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# PAPERS, REPORTS, DISCUSSIONS, AND MEMOIRS

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# TECHNICAL SESSIONS

OCTOBER 9, 1929—10:40 A. M. TO 12:30 P. M.

## DEVELOPMENT OF THE BOSTON METROPOLITAN DISTRICT

By Frederic H. Fay,* M. Am. Soc. C. E.

### A Unique District

The City of Boston is the center of a Metropolitan Area comprising upward of forty separate municipalities so closely knit together, both physically and by the bond of common interests, that they form virtually one large community. Around other large cities of the country, the diffusion of population from congested centers over neighboring suburban areas is of comparatively recent occurrence, and generally these cities have expanded in territory with the spread of population. Boston, always small in area, has had no substantial territorial expansion in recent years; furthermore, it is unique in that for more than half a century the population of the outside territory, of which it is the business center, has outnumbered that within its city limits. The Boston Metropolitan District, therefore, of necessity was the pioneer in meeting and solving problems of common interest to many municipalities, and here metropolitan agencies have been developed both for a longer time and to a greater extent than in any other metropolitan area of the United States.

### Extent and Character

The Boston Metropolitan District at present has a population of 2 000 000, or more, of which approximately 800 000, or 40%, reside within the limits of the city itself, and about 1 200 000, or 60%, live within the 48 cities and towns which, together with Boston, constitute the Boston Metropolitan District as defined by the U. S. Bureau of the Census.

Massachusetts as a whole is, of all the States of the Union, second only to Rhode Island in density of population. If its present estimated population of approximately 4 400 000 were distributed evenly over the 8 039 sq. miles of land area of the State, each square mile would contain about 545 inhabitants. The distribution is far from even, however, and 47% of the State's entire population is found living on 7% of the State's area in and around Boston.

Massachusetts is essentially a State urban in character. For two generations it has led all the States of the Union in the number of its cities having

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25 000 population, or more. It is exceeded only by Pennsylvania in the number of communities of 10 000, or more. Most of these urban municipalities are found in the congested area surrounding Boston; while they are independent political units, they are so closely knit together that they form virtually one large community of which Boston is the center.

MUNICIPALITIES COMPRISED WITHIN THE SEVERAL BOSTON METROPOLITAN DISTRICTS, 1929.

## DEVELOPMENT OF METROPOLITAN AREAS

Within the present generation new and increased facilities for local transportation have enabled the population from the more thickly settled districts of the larger cities to spread out over adjacent and more open suburban territory. Two transportation agencies have contributed largely to this result: The electric street railway, which was first introduced about forty years ago; and the automobile, which has come into such extensive use within the past fifteen or twenty years.

* * * * * * *

The rapid growth of population of suburban territory has affected different cities in different ways. In most cases it has resulted in the territorial growth of the cities themselves by annexation. Chicago, Ill., is an example of a city which has grown in area nearly as rapidly as the growth in population of the urban district of which it is the center. In 1920, 85% of the

entire population of the Chicago Metropolitan District was found within the city limits, and, to-day, it is probable that the city contains more than 80% of the population of its Metropolitan Area.

* * * * * * *

For more than half a century the Boston District has maintained the position which it holds to-day, that of the fourth largest urban district of the country, being exceeded only by New York, Chicago, and Philadelphia, Pa. Of all the large cities, Boston was the first to develop as the center of an important metropolitan territory and the Boston Metropolitan District, therefore, was the pioneer metropolitan district of the country.

PRINCIPAL METROPOLITAN DISTRICTS OF UNITED STATES CITIES IN 1920.*

| Metropolitan District. | Area, in square miles. | Population. | Percentage of distribution. | Population density per square mile. | Percentage increase, 1910-1920. |
|---|---|---|---|---|---|
| NEW YORK, N. Y.: | | | | | |
| Metropolitan District | 1 174.82 | 7 910 415 | 100.0 | 6 733 | 20.5 |
| In city proper | 299.00 | 5 620 048 | 71.0 | 18 796 | 17.9 |
| Outside | 875.82 | 2 290 367 | 29.0 | 2 615 | 27.2 |
| CHICAGO, ILL.: | | | | | |
| Metropolitan District | 733.70 | 3 178 924 | 100.0 | 4 333 | 29.4 |
| In city proper | 192.78 | 2 701 705 | 85.0 | 14 014 | 23.6 |
| Outside | 540.92 | 477 219 | 15.0 | 882 | 76.3 |
| PHILADELPHIA, PA.: | | | | | |
| Metropolitan District | 755.37 | 2 407 234 | 100.0 | 3 187 | 21.4 |
| In city proper | 128.00 | 1 823 779 | 75.8 | 14 248 | 17.7 |
| Outside | 627.37 | 583 455 | 24.2 | 930 | 34.3 |
| BOSTON, MASS.: | | | | | |
| Metropolitan District | 570.43 | 1 772 254 | 100.0 | 3 107 | 15.7 |
| In city proper | 43.55 | 748 060 | 42.2 | 17 177 | 11.6 |
| Outside | 526.88 | 1 024 194 | 57.8 | 1 944 | 19.0 |
| PITTSBURGH, PA.: | | | | | |
| Metropolitan District | 659.43 | 1 207 504 | 100.0 | 1 831 | 16.9 |
| In city proper | 39.87 | 588 343 | 48.7 | 14 756 | 10.2 |
| Outside | 619.56 | 619 161 | 51.3 | 999 | 24.0 |
| DETROIT, MICH.: | | | | | |
| Metropolitan District | 273.83 | 1 165 153 | 100.0 | 4 255 | 126.6 |
| In city proper | 77.87 | 993 678 | 85.3 | 12 761 | 113.3 |
| Outside | 195.96 | 171 475 | 14.7 | 875 | 254.9 |
| ST. LOUIS, MO.: | | | | | |
| Metropolitan District | 308.99 | 952 012 | 100.0 | 3 081 | 14.9 |
| In city proper | 61.00 | 772 897 | 81.2 | 12 670 | 12.5 |
| Outside | 247.99 | 179 115 | 18.8 | 722 | 26.4 |
| CLEVELAND, OHIO: | | | | | |
| Metropolitan District | 232.57 | 925 720 | 100.0 | 3 980 | 48.7 |
| In city proper | 56.39 | 796 841 | 86.1 | 14 130 | 42.1 |
| Outside | 176.18 | 128 879 | 13.9 | 732 | 108.2 |
| SAN FRANCISCO—OAKLAND, CALIF.: | | | | | |
| Metropolitan District | 447.66 | 891 477 | 100.0 | 1 991 | 29.8 |
| In (San Francisco) proper | 42.00 | 506 676 | 56.8 | 12 062 | 21.5 |
| In (Oakland) proper | 45.70 | 216 261 | 24.3 | 4 732 | 44.0 |
| Outside | 359.96 | 168 540 | 18.9 | 468 | 40.7 |
| LOS ANGELES, CALIF.: | | | | | |
| Metropolitan District | 1 299.38 | 879 008 | 100.0 | 676 | 89.1 |
| In city proper | 365.68 | 576 673 | 65.6 | 1 577 | 80.7 |
| Outside | 933.70 | 302 335 | 34.4 | 324 | 107.6 |

* Data compiled from U. S. Bureau of the Census.

Being a pioneer, the Boston Metropolitan Territory was the first to be confronted with problems which were of common interest to many closely knit communities and required concerted action by these communities for their solution. As complete voluntary co-operation between communities was not possible of accomplishment, the State, as the sovereign power, was forced

to step in and to establish among these communities, by appropriate legislation, what were the first legally constituted metropolitan districts in the country.

Separate metropolitan districts composed of different groups of communities were established by the State for different purposes, but no one of the several districts thus established corresponds exactly with the Boston Metropolitan District as defined by the U. S. Bureau of the Census.

* * * * * * *

## Development of Metropolitan Sewerage District*

The first of the legally constituted Metropolitan Districts of Greater Boston was the Metropolitan Sewerage District, created by legislative enactment in 1889. The problem of drainage for the communities clustered about the city early arose because of the topographical character of the Boston Basin which is drained by three rivers, the Mystic, the Charles, and the Neponset, all of which empty into Boston Harbor along the water-front. As Boston was the largest, and the central, community of this drainage district, and as its sewerage problem overshadowed those of the other communities, that city was forced to build a comprehensive municipal system of sewerage before the Metropolitan Sewerage District was established.

*Boston's Early Sewers.*—Boston was first settled in 1630. The original town was located on a small peninsula, almost an island, connected with the mainland by a narrow neck. Beacon Hill, on which the State House is now located, was the central eminence of this small natural peninsula which lay to the east of present Charles Street and Broadway, within what is now known as the down-town business section and the North End. Topographically, the city has undergone radical changes. Most of the original small hills have been leveled. Beginning early in the Nineteenth Century the need for increased area for the town was met by filling the flats along the shore frontage, notably in the Back Bay, until, to-day, not a foot of Boston's original shore line remains.

When the first sewer was built is not known, but it was earlier than 1700, for in 1701, when the population was about 8 000, the nuisance created by frequent digging up of streets to lay new sewers and to repair those previously built had become so great as to lead to the passage of a regulatory ordinance at a Town Meeting. Up to this time, and in fact for a century and a quarter thereafter, until Boston had been incorporated as a city, all sewers were built by private enterprise. When some energetic householder on any street decided that a sewer was needed there, he persuaded such of his neighbors as he could to join him in building a street drain. Having obtained permission to open the street, or perhaps having neglected this preliminary, they built such a structure as was thought necessary on the shortest line to tide-water.

The filling of the flats along the shore frontage necessitated the extension of these old sewers the outlets of which would have been cut off. The exten-

* This account is introductory and supplementary to a more complete treatment given subsequently under the title, "Sewerage in the Boston Metropolitan District," by X. Henry Goodnough, M. Am. Soc. C. E., p. 2509.

sions in the filled areas were built with little or no slope, and, in some cases, the sewers actually ran up hill. The shore outlets were almost invariably below high-tide level and were provided with tide-gates.

* * * * * * *

Under certain conditions of the atmosphere, especially on summer evenings, a well-defined sewage odor would extend over the whole South and West Ends of the city proper. This was the condition existing at the middle of the Nineteenth Century and until the completion of the Boston Main Drainage System in 1884-85.

*Boston Main Drainage System.*—The Massachusetts State Board of Health, the first State Board in the country, was established by legislative enactment in 1868. While it had State-wide jurisdiction, the principal reason for its creation was the unsanitary condition resulting from the concentration of population in the drainage basin in and around Boston. The State Board of Health early considered, and urged action upon, the drainage problem of Boston and its immediate surroundings.

Spurred to action by the City Board of Health, Boston set up a municipal commission in 1875 to study the drainage problem. The report of this Commission recommended two systems of main and branch intercepting sewers, serving territories on both sides of the Charles River. The northerly system, which was to serve not only the Charlestown and East Boston Sections of Boston, but also portions of Cambridge, Somerville, Everett, Chelsea, and Winthrop, was not adopted at the time because of the failure of Boston to secure the voluntary co-operation of the other communities in which much of this system lay. The southerly system, planned to serve the city proper and parts of Brookline and of the Roxbury, Dorchester, and South Boston Sections, lay almost entirely within the city limits and was substantially that built immediately thereafter and known as the Boston Main Drainage System.

Having secured the necessary legislative authorization, the City built the Boston Main Drainage System between 1876 and 1885. It consists of intercepting sewers along the water-fronts of the city proper and South Boston, connected with, and taking all flow other than heavy storm flow from, the old sewers which formerly had discharged directly into tide-water all along these water-fronts. * * * This intercepting sewer system of Boston was the first great undertaking of its kind in the country.*

*Investigations Leading to Establishment of the Metropolitan Sewerage District.*—While the Boston Main Drainage System took care of the greater part of the nuisance caused by the discharge into the tidal waters around Boston of sewage from that city, it did not remedy the situation caused by other communities particularly those to the north of the Charles River.

Voluntary co-operation by interested municipalities being ineffectual to meet this common drainage problem, the State intervened and, by legislation passed in 1884, created the Massachusetts Drainage Commission.

* * * * * * *

*Further details of this system may be found in the subsequent paper on "Sewerage in the Boston Metropolitan District," by X. Henry Goodnough, M. Am. Soc. C. E., p. 2509.

The report of the Massachusetts Drainage Commission, made public at the close of 1885, clearly demonstrated the existing unsanitary conditions in, and the need of common action in the solution of drainage problems by, communities surrounding the City of Boston. The public mind was not satisfied, however, that the answer to the problem had been found in the recommendations of the Drainage Commission. The Legislature directed the State Board of Health to carry the investigation further and, in particular, to consider the question of disposal of sewage in the most thickly populated sections of the Mystic and Charles River Valleys. The State Board of Health attacked the problem in a most thorough manner, and its report to the Legislature in 1889 is a noteworthy document. The Board made exhaustive investigations and experiments regarding methods of sewage disposal by filtration, by chemical precipitation, and by discharge into tide-water.

As the result of its investigations the State Board of Health recommended a comprehensive system of sewerage works, with discharge into Boston Harbor south of Deer Island, for communities north of the Charles River and in the Mystic River Valley, this system being designated as the North Metropolitan System. For the disposal of sewage from certain municipalities in the lower valley of the Charles River, namely, Waltham, Newton, Watertown, Brookline, and the Brighton District of Boston, the Board recommended another system of intercepting sewers to be connected with the Boston Main Drainage System discharging into Boston Harbor at Moon Island.

*Legislation Establishing Metropolitan Sewerage District.*—The exhaustive report of the State Board of Health, made in 1889, was the final step and resulted in the passage by the Legislature that year of the Act establishing the Metropolitan Sewerage District and creating the Metropolitan Sewerage Commission to build, maintain, and operate comprehensive sewerage systems to care for the sewage of the District. Thus, there came into existence, in 1889, the first legally constituted Boston Metropolitan District.

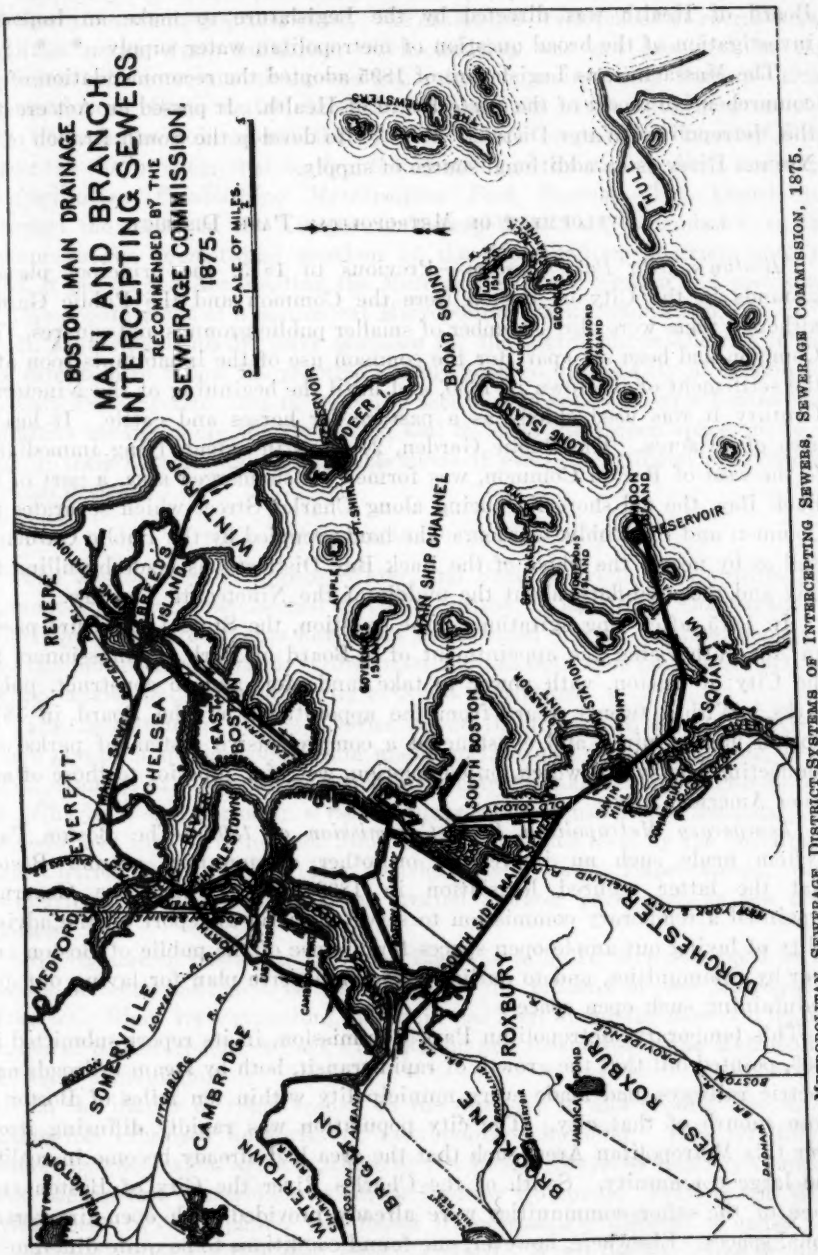
## Development of Metropolitan Water District*

In 1652, a private company was chartered by the General Court of Massachusetts Bay Colony under the title of the "Water Works Company" * * *.

The first organized attempt to induce Boston to construct a system of water supply at public expense was made in 1825, three years after the town was incorporated as a city * * *. There was considerable opposition * * *. Finally, popular demand became so strong that legislation was secured in 1846 * * *. Lake Cochituate water was first furnished in 1848 when the population of the city was 127 000.

In the early Seventies it became apparent that Lake Cochituate was inadequate to meet the needs of the growing city and that additional water must be obtained. After investigation the City decided to secure water from the Sudbury River water-shed lying immediately west of Lake Cochituate * * *. This water was first introduced into Boston in 1878 * * *.

* This interesting account is augmented by a more complete history to be found in the subsequent paper on "Metropolitan Water Supply," by Frank E. Winsor, M. Am. Soc. C. E., p. 2496.




METROPOLITAN SEWERAGE DISTRICT-SYSTEMS OF INTERCEPTING SEWERS, SEWERAGE COMMISSION, 1875.

By 1893 the problem of water supply had become most pressing, not only for the City of Boston, but also for most of the cities and towns near-by. The Board of Health was directed by the Legislature to make an important investigation of the broad question of metropolitan water supply * * *.

The Massachusetts Legislature of 1895 adopted the recommendation of this comprehensive report of the State Board of Health. It passed an Act creating the Metropolitan Water District * * * to develop the South Branch of the Nashua River as an additional source of supply.

### Development of Metropolitan Park District

*Boston's City Park System.*—Previous to 1875, the principal pleasure grounds in the City of Boston were the Common and the Public Garden, although there were also a number of smaller public grounds and squares. The Common had been set apart for the common use of the inhabitants soon after the settlement of the town in 1630, and until the beginning of the Nineteenth Century it was used chiefly as a pasture for horses and cattle. It has an area of 48 acres. The Public Garden, 24 acres in extent, lying immediately to the west of Boston Common, was formerly a submerged area, a part of the Back Bay, the old shore line being along Charles Street which separates the Common and the Public Garden. The land occupied by the Public Garden as well as by nearly the whole of the Back Bay District was made by filling the Bay and the mud-flats, about the middle of the Nineteenth Century.

In 1875, after long agitation of the question, the State Legislature passed an Act authorizing the appointment of a Board of Park Commissioners for the City of Boston, with power to take lands for, and to construct, public parks. Within twenty years from the appointment of the Board in 1875, Boston had acquired and constructed a comprehensive system of parks and connecting boulevards which, on completion, were far superior to those of any other American city.

*Temporary Metropolitan Park Commission of 1892.*—The Boston Park System made such an impression on other communities around Boston that the latter secured legislation in 1892 under which the Governor appointed a temporary commission to serve one year to report on the advisability of laying out ample open spaces for the use of the public of Boston and near-by communities, and to outline a comprehensive plan for laying out and maintaining such open spaces.

This temporary Metropolitan Park Commission, in its report submitted in 1893, pointed out that the growth of rapid transit, both by steam railroads and electric railways, had made every municipality within ten miles of Boston a close suburb of that city. The city population was rapidly diffusing itself over this Metropolitan Area, such that the area had already become in reality one large community. South of the Charles River the City of Boston and some of the other communities were already provided with open-air recreational spaces. Elsewhere, however, one found conditions to be quite different—miles of thickly settled territory with little provision for such open spaces and in many communities no lands available for recreational use. Socially,

this great district was one, and its recreational needs should be met as a unified problem, otherwise its natural development would surely be hampered.

Two solutions of this problem were possible, one, consolidation of municipalities into a single political entity; the other, the development of recreational facilities under State authority. Consolidation was not practical, as the sentiment of the communities was generally for maintaining their political independence. State action, however, was deemed to be entirely practical. The Commission pointed out that, of the world's large cities, London alone presented the problem that confronted Boston and its surrounding territory.

*Legislation Establishing Metropolitan Park System.*—The Legislature accepted the report of the Temporary Metropolitan Park Commission as the answer to the recreational problem of the Metropolitan District, and, in 1893, it passed the Act creating the Metropolitan Park System and providing for the appointment of a permanent Metropolitan Park Commission.

In acquiring lands for park purposes for the Metropolitan District it was the intention to follow the practice already adopted by the City of Boston for its City Park System, that is, to aim to secure open spaces at once before communities became more thickly settled and to leave these open spaces more or less in their natural state until such time—if ever—as artificial development seemed desirable.

## Other Metropolitan District Activities

The Metropolitan Sewerage, Water, and Park Districts are the three permanent and legally established districts created by legislative enactment, for which public improvements have been constructed for the common benefit of the communities concerned. Other metropolitan activities have been, and are being, carried out under authority of State legislation.

The improvement of the Charles River Basin effected about twenty-five years ago by a special commission under authority of State legislation, is the chief example of a special metropolitan constructional activity. This part of the Charles River, formerly a tidal estuary with offensive mud-flats, has been transformed into a fresh-water basin held at constant level by the building of the Charles River Dam. The Charles River Basin Improvement, while primarily a sanitary measure, has created a water park such as few cities possess.

By legislation passed in 1929, the Basin is to be further developed to add to its attractiveness and its usefulness to the citizens of the Metropolitan District. Since its completion, the Basin has been under the care and maintenance of the Metropolitan Park Commission and its successor, the Metropolitan District Commission.

In the matter of rapid transit in the territory in and around Boston, the State has exercised authority of metropolitan character. This territory is served by two transit systems, those of the Boston Elevated Railway Company and the Eastern Massachusetts Street Railway Company. Prior to the World War these two Companies had found it increasingly difficult to make both ends meet under the fare restrictions imposed. When the war came, bringing in its train the rapid increase in the general scale of prices, the situation

became increasingly acute and, by 1918, both Companies were faced with the alternative of either securing increased revenue or going into bankruptcy. Adequate transportation was a matter of deep concern to the communities depending on these two companies for service. Public ownership was not favored. Instead of public ownership, the Legislature of 1918 passed two Public Control Acts, one applicable to the Boston Elevated Railway System, the other to the Eastern Massachusetts Street Railway Company.

In each case, there was created a Board of Trustees, appointed by the Governor, which took over the control and the operation of the system as a matter of public necessity. Former fare restrictions were abolished. Each Board of Trustees was directed to operate its system at cost; and was given power to establish rates of fare sufficient to pay the expense of operation and upkeep, to meet interest and retirement charges of bonds and preferred stock, and also to pay dividends on common stock of the Company at rates fixed by the Public Control Act. Thus, these two transportation systems were taken out of the hands of the stockholders and put under the control of public officials where the control remains to-day, although the question is now under discussion as to what the future arrangement shall be. Thus far, public control has been of distinct benefit to the Metropolitan District and has been the means of saving the situation which had become most critical in 1918.

The Metropolitan Planning Division was established by legislation passed in 1923. Although a division within the Metropolitan District Commission, it is broadly representative and of the seven commissioners in charge, three are appointed at large by the Governor and the others are official representatives of the State Departments of Public Works and of Public Utilities, the Metropolitan District Commission, and the Transit Department of the City of Boston. The Metropolitan Planning Division is charged primarily with the duty of investigating and making recommendations as to transportation service and facilities within the Metropolitan District. In no sense does it supplant local planning agencies, nor does it deal with projects exclusively local in character; it aims to secure co-ordination on broader matters affecting the District as a whole.

* * * * * * *

### Financing Metropolitan Improvements

The State of Massachusetts has loaned its credit in the first instance and has issued the bonds necessary to meet the cost of metropolitan developments. In no case, however, have these developments been treated as State-wide undertakings, nor has the work of construction been carried out by established State departments; instead, the State has created a special agency to handle each separate metropolitan problem.

Altogether, the State has issued considerably more than $100 000 000 of bonds for metropolitan improvements. The cost of maintaining and operating these improvements is met by legislative appropriation from the State Treasury. By these transactions, the State is not out of pocket, however, as interest and retirement of bonds and the cost of maintenance and operation are reimbursed to the State by direct tax on the municipalities of each of

the several Metropolitan Districts. The method of apportionment of this tax is variable, but is usually based either on the population or on the assessed valuation, or both, of the communities benefited.

### Why Not Greater Boston by Annexation?

To-day, 2 000 000 people live in the forty-nine cities and towns designated by the U. S. Bureau of the Census as comprising The Boston Metropolitan District. To a very large extent these several communities are bound together by common interests in matters such as water supply, sewerage, and drainage, public health, parks, transportation, postal facilites, and public safety. The question naturally arises why, with this large and varied community of interests, do not these several municipalities unite with the City of Boston under one central municipal government? Thus, "Real Boston" would become a single city and would be recognized for what it really is instead of remaining as it is to-day a central city surrounded by upward of forty satellite communities. The added prestige which Boston would gain by such public recognition of her rightful place among the really large cities of the world has long been urged as one of the reasons for the annexation of these neighboring municipalities. The City of Boston would then include nearly one-half the population of the entire State.

The question of annexation is largely one of psychology. Boston is willing, but in the case of many of the smaller cities and towns local pride, historical tradition, and differences in racial and political complexion, all weigh against outright annexation. The communities are jealous of their existence as separate entities, although willing to co-operate with each other in matters of mutual concern. There is no present indication that "Real Boston" will result from the annexation to Boston of forty or more municipalities; instead, the Boston Metropolitan District, in its present and possibly in enlarged fields of activity, seems likely to function for many years to come.

## ADMINISTRATION OF THE BOSTON METROPOLITAN DISTRICT

By Davis B. Keniston,* Esq.

Unlike most large cities, Boston has not extended its corporate limits by annexing the surrounding territory usually considered as suburban areas. Instead, the cities and towns lying within this area have retained their local governments and political control.

The metropolitan area of Boston is usually considered as including those cities and towns any part of the territory of which is within ten miles of the State House. The Metropolitan District as it is composed for the exercise of the Metropolitan Systems controlled by the Metropolitan District Commission includes thirty-five such municipalities and eight situated outside

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this area. The population of Boston in round figures is 800 000, while that of the entire area including Boston is 1 800 000. These are the people who contribute to these systems and are directly benefited by their advantages, although many times this number from all sections of the State and Nation are privileged to, and do, share these benefits.

*Scope of Commission.*—The Metropolitan District Commission exercises the functions formerly under the jurisdiction of the Metropolitan Water and Sewerage Board and the Metropolitan Park Commission. These functions are defined in general terms by the statutes creating these boards, as follows: "to construct, maintain and operate main sewers and disposal works"; "a system of water works"; "to acquire, maintain and make available open spaces for exercise and recreation"; and "to connect any road, park, way or other public open space with any part of the cities and towns of the district by suitable roads and boulevards."

There are four Metropolitan Districts, two Sewer Districts, North and South, a Water District, and a Parks District. The Sewer and Water Districts include only those cities and towns directly served by the sewer and water systems and comprise in each instance only a part of the municipalities within the Metropolitan Area. The Parks District includes forty municipalities, or practically all those included in the entire area.

*Methods of Organization.*—The first of these activities undertaken was the sewerage system and the Act passed in 1889 authorizing its construction created a commission of three members to be known as the Board of Metropolitan Sewerage Commissioners. The Governor appointed the members, who were to be "three able and discreet men, inhabitants of the Commonwealth." The Board chose one of its members as Chairman, elected a Clerk not a member of the Board, and appointed such agents, officers, and servants as it deemed necessary to carry out the purposes of the Act and determined their duties and compensation, and removed them at pleasure. The members held office for three years, their terms expiring in succeeding years. The Act specified the cities and towns that were included in each of the systems, and no provision was made for enlarging the districts. Other municipalities have been added by special legislative acts. There are now eighteen cities and towns in the North District and fourteen in the South District.

The Metropolitan Water System was authorized in 1895. Here, again, the Governor was given authority to appoint three Commissioners who were to constitute the Metropolitan Water Board, to hold office for the same terms as those of the Sewerage Board, and was to designate the member to serve as Chairman. One member was to be a resident of Boston, one member a resident of one of the other cities and towns in the District, and the third, any citizen of the Commonwealth. The Board was to appoint an engineer, a secretary, and such other agents, officers, and clerks, as it deemed necessary. The Water District was first made up of those cities and towns that were to be furnished with water. It was provided by the Act that any other city or town, any part of which was within ten miles of the State House, could be admitted to the District upon such terms and conditions as the

Commission should determine. Several have been added, and the District now includes twenty cities and towns.

The sewerage systems had been largely constructed by 1901 when the Sewerage Board was combined with the Water Board under a new Commission called the Metropolitan Water and Sewerage Board, to which was transferred all the powers, rights, duties, and obligations of the other two Boards. It was composed of three members similarly appointed, the Governor designating the Chairman. No restriction was placed on the qualifications of members for appointment.

The Metropolitan Parks System was started in 1893 and placed in charge of a Metropolitan Park Commission of five members appointed as in the other cases by the Governor without restriction as to qualification for membership, to hold office for terms of five years. The members were to serve without compensation and did so until 1901, when salaries were attached to the positions. The Board chose its Chairman and was empowered to appoint a clerk or secretary and such other employees as it deemed necessary, including a suitable police force.

*District Commission Created.*—By a Constitutional Amendment adopted in 1918 all State boards and commissions were required to be organized into not more than twenty-one, and in carrying out this re-organization in 1919, the Metropolitan District Commission was created, to take over all the powers, duties, and responsibilities of the Metropolitan Park Commission and the Metropolitan Water and Sewerage Board. It consists of a Commissioner and four Associate Commissioners who must be residents of the District and at least one must be a resident of Boston. The Commissioners are appointed by the Governor to hold office for terms of five years, the term of one Commissioner expiring each year.

The Act creating the Commission provides that the Commissioner shall organize the Commission in such divisions as he may from time to time determine, and, with the approval of the Governor and Council, he may appoint a director of each division to have charge of its work. The Commission is authorized to employ a secretary, engineering chiefs, and such other officers and employees, including a suitable police force, as the work may require.

In accordance with these provisions the general work of the Commission is carried on by a secretary, clerical, and bookkeeping forces. The construction, operation, and maintenance of the Metropolitan Systems have been divided into four divisions: Parks, Park Engineering, Water, and Sewerage, each in charge of a director. The Directors of Park Engineering, Water, and Sewerage also serve as Chief Engineers of their Divisions.

*Sewerage.*—The Director and Chief Engineer of the Sewerage Division is responsible for the construction, operation, and maintenance of both sewerage systems. The other employees of this Division consist of an Engineering Staff and a small clerical force, whose work is general to both systems, and of the operating and maintenance forces of the separate systems. The average number of persons employed in this Division is 225 and the annual cost of maintenance is about $575 000.

*Water Supply.*—The construction, operation, and maintenance of the Water System is in charge of a Director and Chief Engineer, with an engineering and clerical force. The operation and maintenance of this system is divided into four sections, each in charge of a superintendent. The Distribution Section has charge of the distribution system, its mains and reservoirs, and other properties; the Sudbury Section is composed of the Sudbury, Southern Sudbury and Cochituate water-sheds, their storage reservoirs, and aqueducts; the Wachusett Section covers that water-shed and reservoir, with the aqueduct and properties within it; and the Pumping Section includes all the pumping stations within the distribution area. The Water Division employs an average force of about 362, and its maintenance costs are about $800 000 per year.

*Parks.*—The work connected with the Metropolitan Parks and Boulevards is divided into two divisions, Parks and Park Engineering. The Parks Division maintains and controls the reservations and parkways and these are grouped according to location into six divisions, each under a superintendent. The Superintendent is also a Police Captain, has charge of both the labor and police forces, and is responsible for the care and control of his Division. One of these Superintendents is designated as a Captain in charge of the general supervision of the Police Department and recommends the police assignments. The Parks Division has an average maintenance force of 377 and a permanent police force of 180.

The Parks Engineering Division, under the supervision of the Director and Chief Engineer, has charge of all engineering work for construction and maintenance. It has direct charge of all construction except such smaller repairs or construction as may be handled by the maintenance forces, and to these it gives such engineering assistance as may be required. It oversees all building and entrance construction; checks property lines to avoid encroachments; recommends locations for municipal and public utility structures; and has direct charge of the maintenance of all bridges, locks, and draws. It employs an average force of 43 engineers and 57 other employees. The maintenance cost of the Parks Systems, including special appropriations for improvements, is about $1 200 000.

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*Operation of District Commission.*—The Metropolitan District Commission, although its activities are exercised for the metropolitan cities and towns and not for the entire State, is a State Commission and functions in all respects as such. Title to all property forming a part of the various systems is in the State; the Commission is subject to the same supervision and control as other State Commissions; its employees, except those specially exempted, are under civil service laws; appointments are subject to approval by the Commission on Administration and Finance; and its expenses are subject to budget estimates and legislative appropriation. The Commission makes its annual recommendations and reports to the Legislature, as do other State Commissions.

The Commission has general authority to operate and maintain the several systems and the properties comprising them. The general appropriations made for these purposes do not permit of any extensive additions or improvements. Authority for such work, as well as changes or amendments to existing laws affecting the district activities, are the subject of recommendations and petitions to the General Court and follow the same procedure as other legislative matters.

*Financial Aspects.*—All the expenses incurred in the construction, operation, and maintenance of the Metropolitan Systems are paid in the first instance by the State and are then assessed back on the municipalities included in the Metropolitan District for which the expense is incurred. The proportions or percentages of the total cost which each municipality is to pay are determined by the Commission in the manner prescribed by law and certified by it to the State Treasurer. The State Treasurer determines the total amount of appropriations, interest, sinking fund, and serial bond payments required, and assesses that proportion of the total, according to the percentages, upon each of the municipalities.

When the different metropolitan activities were first created, it was provided that, except in the case of the Water District, the proportions in which each of the cities and towns in the District should pay money into the Treasury of the Commonwealth were to be determined by three commissioners to be appointed by the Supreme Judicial Court in such manner as the Commissioners "shall deem just and equitable." The Wachusett Act, passed in 1895, creating the Metropolitan Water District, provided that the proportion of the total cost for the construcion and maintenance of the system which the City of Boston should pay, should be based on the percentage of its valuation to the entire valuation of the Water District, and that the other cities and towns should contribute the remainder on a proportion based one-third on valuation and two-thirds on population. In 1905, these proportions were changed so far as the other cities and towns were concerned by substituting "consumption" of water for "valuation" of property. In 1906, Boston's percentage was determined in the same manner as that of other municipalities, and this proportion, based one-third on valuation and two-thirds on consumption, for assessing both construction and maintenance charges, has since been used.

*Proportioning Costs.*—In the Sewerage Districts the first Commission appointed fixed a proportion based on valuation for construction costs and on population for maintenance cost. The second Commission appointed fixed the same method for determining apportionments, and, in 1906, this basis was enacted into law and has since remained unchanged.

The first Metropolitan Parks legislation provided that the proportions for this System should be determined by Commissioners appointed by the Supreme Judicial Court every five years. The basis fixed by the first Commissioners appointed was on a percentage of taxable valuations for interest and sinking-fund requirements, and upon a percentage of taxable valuations and populations for maintenance of reservations, and the District's share

of the cost of boulevards. This method was followed by succeeding Commissions until 1919 when, at the request of the municipalities concerned, it was established by law as the basis for the park assessments.

The amounts to be paid by the municipalities as finally determined by the State Treasurer are included in the other State assessments against the cities and towns and are collected as a part of the State tax. The State pays one-half the cost of construction and maintenance of the boulevards, and this contribution is now met from the Highway Fund collected from automobile registration fees and the gasoline tax.

All bonds issued for construction costs of the Metropolitan Systems are State bonds on which the State is in the first instance responsible, but is reimbursed for all payments by the Districts. Until about 1924 practically all construction costs were met by new bonds, and the bond issues represented to a large extent the original cost of the Systems. Since that time the cost of fairly substantial improvements has often been met by current appropriations. This has somewhat checked the increase in the bonded indebtedness. Until 1912 all bonds issued were sinking fund bonds, most of them on a forty-year basis, but since that time issues have been serial bonds.

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*Metropolitan Planning.*—In 1923, there was organized by legislative enactment within the Metropolitan District Commission a Division of Metropolitan Planning. This Division is in charge of seven Commissioners, three of whom are appointed by the Governor for terms of five years without compensation, a member from the Department of Public Works, designated by the Commissioner, a member from the Department of Public Utilities, designated by the Chairman, a member from the Metropolitan District Commission, designated by the Commissioner, and an officer of the Transit Department of the City of Boston, designated by the Chairman. The Chairman of the Division is designated by the Governor.

The functions of this Division are to investigate and make recommendations as to transportation service within the District and the co-ordination of highways and other arteries of traffic. Although created as a Division of the Metropolitan District Commission it carries on its work as a separate body. The membership brings together in one group representatives of most of the other bodies the work of which is concerned with such problems. The Division employs permanent engineering and clerical forces and temporarily such additional engineering and consultant experts as its investigations require.

The work of the Division is entirely advisory and recommendatory and requires no operating or maintenance employees. The Legislature refers to this Division for reports on many of the bills relating to transportation within the District; and its recommendations have been the basis for many of the boulevard, highway, and rapid transit improvements authorized for the years since its creation.

*Water Supply Commission.*—When the additional supplies for the Metropolitan Water System were authorized in 1926 to include the Ware and Swift Rivers, there was created for the purpose a special Metropolitan District Water

Supply Commission consisting of the Commissioner of the Metropolitan District Commission as Chairman, and of two Associate Commissioners to be appointed by the Governor.

This Commission was authorized to appoint a Chief Engineer and such engineering, legal, clerical, and other assistance as it deemed necessary. The Commission has organized its work by the appointment of a secretary and office force, a bookkeeper, and an engineering force under a chief engineer. It maintains its chief office in Boston with three field offices. * * *.

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## PARK SYSTEM OF THE BOSTON METROPOLITAN DISTRICT

By Edwin H. Rogers,* M. Am. Soc. C. E.

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After many years of agitation, the Legislature of Massachusetts, in 1892, passed an Act providing for the appointment of a so-called Temporary Board of Metropolitan Park Commissioners, whose term of office was to be one year.

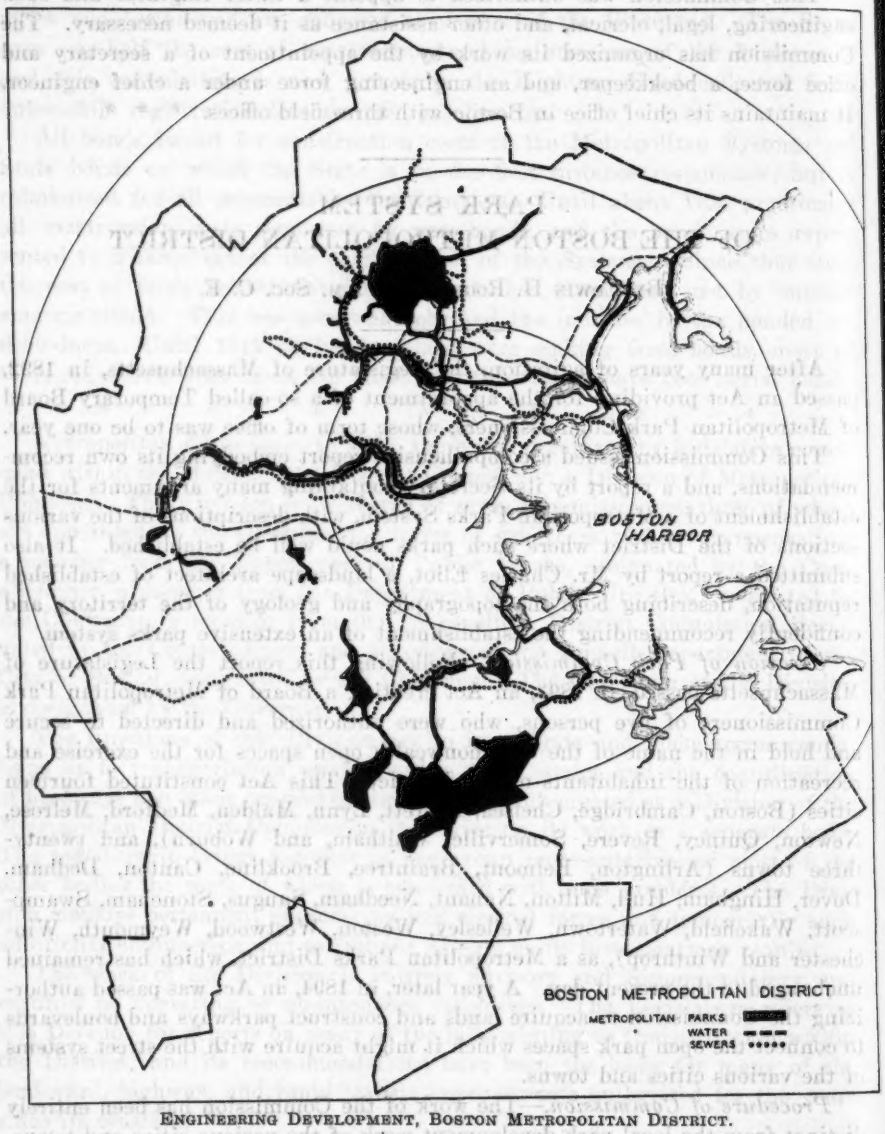
This Commission issued a comprehensive report embodying its own recommendations, and a report by its Secretary containing many arguments for the establishment of a Metropolitan Parks System, with descriptions of the various sections of the District where such parks could well be established. It also submitted a report by Mr. Charles Eliot, a landscape architect of established reputation, describing both the topography and geology of the territory and confidently recommending the establishment of an extensive parks system.

*Creation of Park Commission.*—Following this report the Legislature of Massachusetts passed, in 1893, an Act creating a Board of Metropolitan Park Commissioners of five persons, who were authorized and directed to secure and hold in the name of the Commonwealth open spaces for the exercise and recreation of the inhabitants of the District. This Act constituted fourteen cities (Boston, Cambridge, Chelsea, Everett, Lynn, Malden, Medford, Melrose, Newton, Quincy, Revere, Somerville, Waltham, and Woburn), and twenty-three towns (Arlington, Belmont, Braintree, Brookline, Canton, Dedham, Dover, Hingham, Hull, Milton, Nahant, Needham, Saugus, Stoneham, Swampscott, Wakefield, Watertown, Wellesley, Weston, Westwood, Weymouth, Winchester and Winthrop), as a Metropolitan Parks District, which has remained unchanged to the present day. A year later, in 1894, an Act was passed authorizing the Commission to acquire lands and construct parkways and boulevards to connect the open park spaces which it might acquire with the street systems of the various cities and towns.

*Procedure of Commission.*—The work of the Commission has been entirely distinct from the local park development work of the various cities and towns, but co-ordinated therewith. The District has an area of about 400 sq. miles and, at the time of its establishment, in 1892, had a population of about 900 000 which has more than doubled in the succeeding thirty-seven years.

* Director of Park Eng., Metropolitan Dist. Comm., Boston, Mass.

The Commission organized an engineering force and engaged the well-known landscape architectural firm of Olmsted, Olmsted, and Eliot, as Consultants. It recognized the demand that recreation territory to be acquired

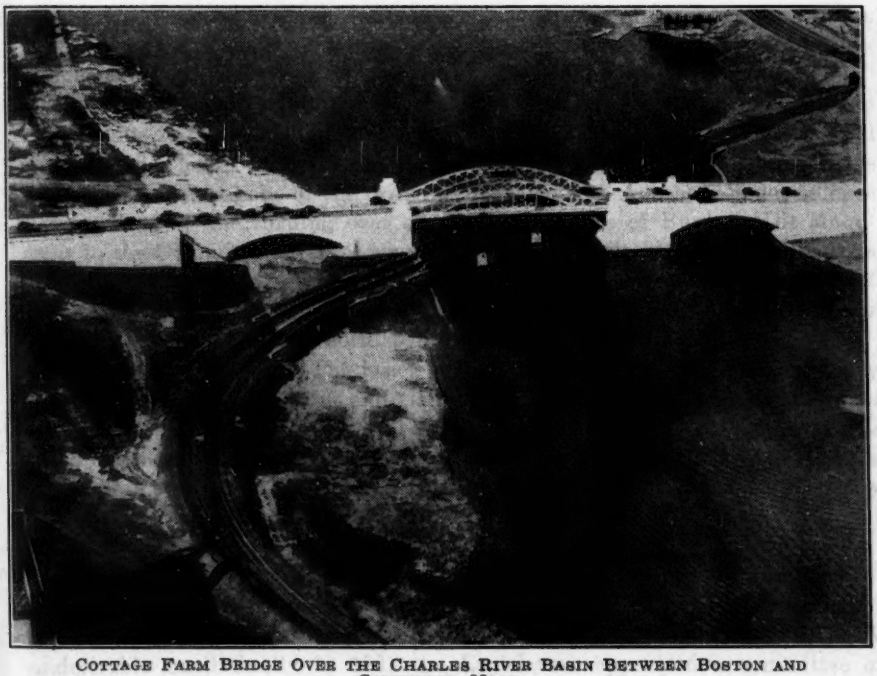



ENGINEERING DEVELOPMENT, BOSTON METROPOLITAN DISTRICT.

should be of varied character, comprising wooded tracts and salt and fresh-water reservations, with good facilities for reaching such localities.

Within two years more than 6 000 acres had been acquired, which has been added to from time to time, and the total area now held is somewhat in excess

SHORE-PROTECTION WORK, QUINCY, MASS.



COTTAGE FARM BRIDGE OVER THE CHARLES RIVER BASIN BETWEEN BOSTON AND CAMBRIDGE, MASS.

of 11 000 acres. Except for the parkway areas, this acreage is largely hilly, wooded, and rocky tracts.

*Beach Development.*—Early attention was given to the ocean frontage in the District. The Revere Beach Section, north of Boston, was acquired in 1895. This locality was popular as a salt-water resort, but exceedingly unattractive. Many cheap buildings were scattered along the beach, a narrow-gauge railroad ran close to the shore, and there were many other undesirable features. The Commission relocated the railway, condemned and removed the poor property, and built a boulevard along the shore, with provision for amusement resorts on the landward side. The demand for bathing was recognized and, in 1897, a large bathhouse was built. It is of interest to note that provision was made for the storage of 1 200 bicycles.

Two years later, in 1899, a considerable section of ocean frontage was acquired at Nantasket Beach in the Town of Hull, and this resort was improved in a manner similar to Revere Beach, but much less work was necessary. Takings of further salt-water frontages in Winthrop, Swampscott, Quincy, and other places soon followed.

*Transportation Improvement.*—In 1899 a speedway was built for the use of pleasure horse-drawn vehicles along what is now Soldiers Field Road adjacent to the Harvard Stadium, and a year later, in 1900, the present trotting track was built. This is now, practically the only suitable place in the vicinity of Boston for the exhibition of trotting horses.

Transportation by other than private horse-drawn vehicles to the park lands was an important item in the early days of the Commission's work, and street railway construction was encouraged as a means of reaching the various parks. Street railway development was popular in those days, and many lines were constructed which have since been abandoned.

By 1905, the increasing use of automobiles rapidly became a real problem on account of the wear and disintegration of the roads caused thereby, and necessitated a heavy program for re-surfacing the park roads with bituminous pavements. In 1919 one of the principal memorials to American independence was acquired by the turning over of the maintenance of Bunker Hill Monument to the Commission. This structure had previously been maintained by a private association with the aid of private contributions. In 1925, a number of bridges crossing the Charles River were turned over to the Commission for care and control, including both span and draw-bridges.

*Charles River Basin.*—The lower portion of the Charles River, particularly between the Cities of Boston and Cambridge, forms a wide basin, and as this is a tidal river for a distance of about eight miles from the harbor, there was much complaint about the condition of that part of the river, owing to the flats which were exposed at low tide. A dam at some location on the river, generally near its entrance to the harbor, had been discussed at least from 1859, and perhaps earlier.

Acts were passed by the Legislature from time to time with a view to providing for the improvement of the lower part of the river by a dam or other means, but nothing was accomplished. Conditions became more and more undesirable until finally in 1901 the Legislature appointed a committee of

three to make a thorough investigation and report. This Committee employed experts in all the subjects relating to the improvement of the Basin and issued a remarkably comprehensive and complete report as the result.

Legislation was passed in 1903 establishing a Charles River Basin Commission, with instructions and authority to build a dam near the harbor and certain embankments along the Basin. This work was completed in 1910 and impounded the water of the Basin at a fixed level at about 8 ft. above mean low water and 2 ft. below the average high water, with locks for navigation and sluices for water control. The care and control of the dam and the embankments and future work on the Basin was then turned over to the Metropolitan Park Commission. The water area of this Basin is about 750 acres.

*Present Status.*—In 1920, the Metropolitan Park Commission was abolished by the Legislature and the Metropolitan District Commission created, which took over the maintenance, operation, and construction of the Metropolitan Sewer Systems and the Metropolitan Water System, thus greatly enlarging its powers and duties.

To sum up, the Commission's development work has taken the form of forestry in the opening up for public use of large wooded tracts, the taking and improvement of certain beach resorts and the building of bathhouses, the construction of parkways and boulevards both inland and along the ocean frontages, the replacement of old bridges over the various streams and rivers with new structures, the acquiring of lands along the river valleys and encouragement of the use of the rivers, the providing of riding and driving facilities for horse lovers, and all in all a broad program of metropolitan recreational and transportation development at a total construction cost of about $30 000 000.

## METROPOLITAN WATER SUPPLY

By Frank E. Winsor,* M. Am. Soc. C. E.

The early history of the struggle to obtain adequate water for a rapidly growing population reads much the same in the case of each of the then principal American cities of 150 years ago, namely, New York, Philadelphia, and Boston.

### Development of Former and Existing Supplies

*Original Works.*—A Water-Works Company was incorporated in Boston in May, 1652, and in 1795 and 1796 an Aqueduct Corporation was formed to bring from any part of the Town of Roxbury (now a part of Boston) into the Town of Boston, and into any street of that town, all such fresh water as it then had or later might have a right to dispose of. The water was brought from Jamaica Pond, now a part of the Park System of Boston. For the first 10 years of operation, this Company paid no dividends, and for a

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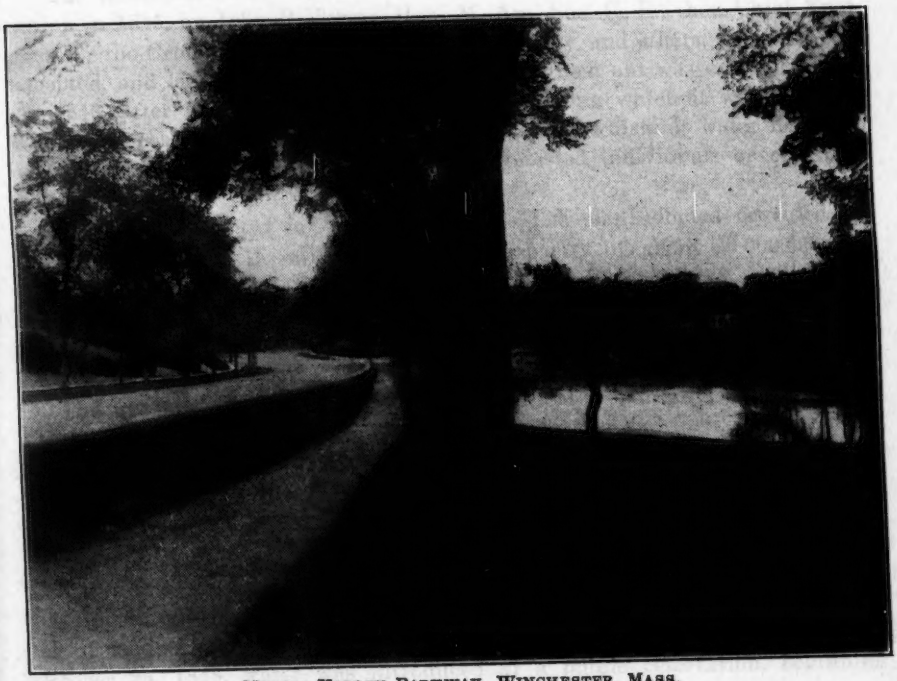
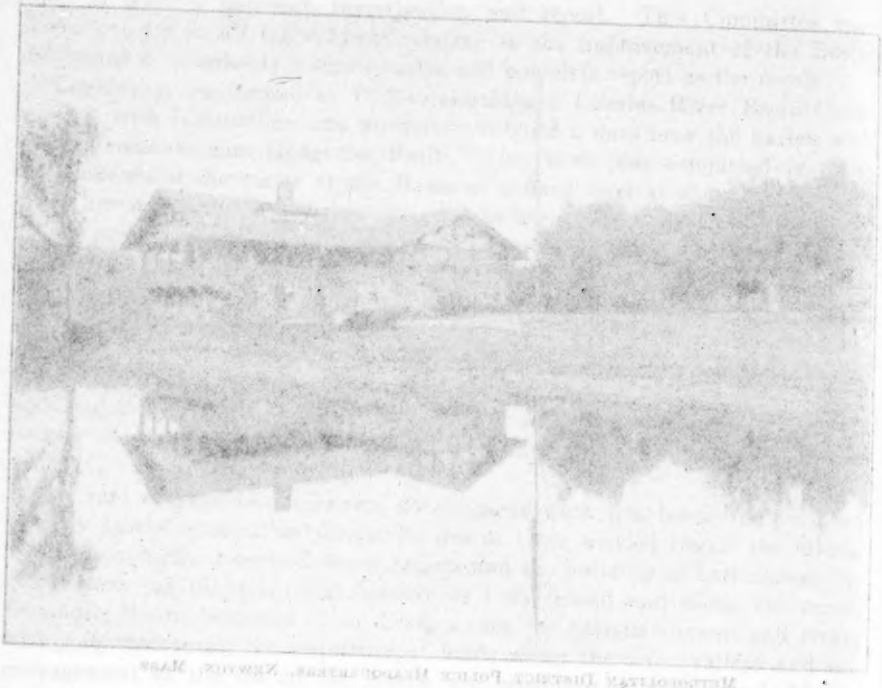
METROPOLITAN DISTRICT POLICE HEADQUARTERS, NEWTON, MASS.



MYSTIC VALLEY PARKWAY, WINCHESTER, MASS.



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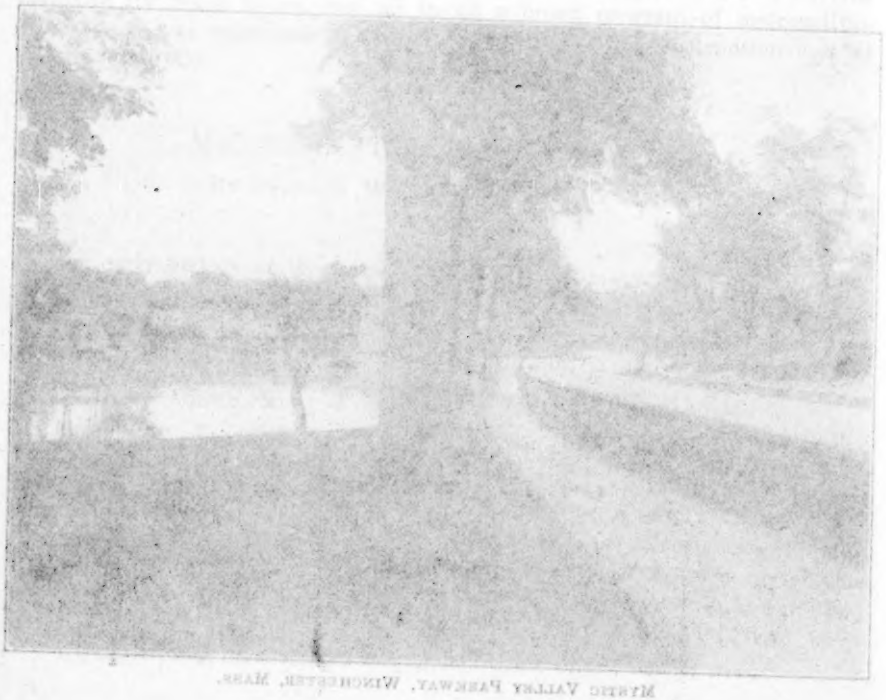


MYSTIC VALLEY PARKWAY, WINCHESTER, MASS.

period of 30 years earned less than 4% per annum on a capital of about $130 000. The price for water furnished to customers was fixed by the General Court.

This Company reached its greatest prosperity about 100 years ago, when it had four main lines of pitch pine log pipes, two of 4-in. bore, and two of 3-in. bore, extending from Jamaica Pond to the then settled part of Boston, with about 15 miles of similar pipe of 1½-in. bore for the distribution system. This Company then supplied 1 500 houses, principally east of Boston Common. In 1840 it laid a 10-in. iron pipe line, about 5 miles in length, from Jamaica Pond to Bowdoin Square.

*Report on Publicly Owned Supply.*—Agitation for a publicly owned water supply began in 1825, at which time a report states that Philadelphia, with a population of 60 000, had a water supply capable of furnishing 3 000 000 gal. daily; that London furnished 29 000 000 gal. daily to a population of about 1 000 000; and that Boston, with a population of 50 000, should have a supply of not less than 1 600 000 gal. daily.

In May, 1834, the Mayor of Boston appointed Col. Laommi Baldwin, an eminent engineer of his time, to make an examination and report, this being the first engineering investigation for a Boston water supply. Five months later, in October, 1834, Colonel Baldwin made a report of seventy-eight pages, said at the time to be "one of the most complete reports on the subject of supplying a city with water that can be found."

The recommendations of this report, after a further engineering investigation in 1845, by John B. Jervis, Hon. M. Am. Soc. C. E., then Chief Engineer for the Croton Water-Works of New York City, and others, were virtually accepted, and legislation providing for carrying them out was enacted March 30, 1846, after twenty-one years of agitation, during which it would appear that every possible and every fantastic scheme of furnishing water had been suggested by would-be experts or by self-interested individuals or organizations.

*Reaching Westward for Boston's Water.*—The plan adopted provided for an aqueduct about 15 miles long, with a capacity of about 20 000 000 gal. daily, leading westerly into an area then sparsely populated, and taking the water supply from what is now known as Lake Cochituate, with a drainage area of 17.6 sq. miles and an available storage of about 2 000 000 000 gal. formed by raising the lake 10 ft.

The water supply policy of Boston and the Metropolitan District for the succeeding eighty years has been to steadily go farther and farther to the west, in order to obtain water from drainage areas sparsely populated and relatively free from contamination, and to store these waters in large reservoirs. It has tended both to increase the dependable yields and to improve the quality of the water.

*Cochituate Supply.*—The first commission of three to carry out the work was appointed May 4, 1846, with salaries fixed at $3 500 per annum. Breaking of ground on August 20, 1846, was attended by elaborate ceremonies. On October 25, 1848, water was introduced at a public celebration, beginning

at the break of day with a salute of 100 guns, followed later by a procession of military, civic, business, and fraternal organizations representing not only Boston, but the State of Massachusetts, and, in many instances, all the New England States. Addresses were made, followed in the evening by an elaborate display of fireworks.

The contract prices for some of the principal items of work on the Cochituate Aqueduct, which is mainly a cut-and-cover masonry structure, egg-shaped, of 6 ft. height, and of 5 ft. maximum width, with two short tunnels, were as follows:

Open cut excavation, per cubic yard.....9 to 20 cents
Cast-iron pipe, per ton of 2 240 lb.....$45.00 to $47.50
Brick delivered, per 1 000.....$5.50 to $7.50

The two tunnels were 2 410 and 1 150 ft. in length, respectively, and had a maximum depth of cover of about 80 ft. Shafts were located about 400 ft. apart.

The total cost of the Cochituate Works, prior to May 1, 1867, which includes the distribution system of the City, was about $6 000 000; and the gross revenue increased from $72 000 in 1849 to $522 000 in 1867.

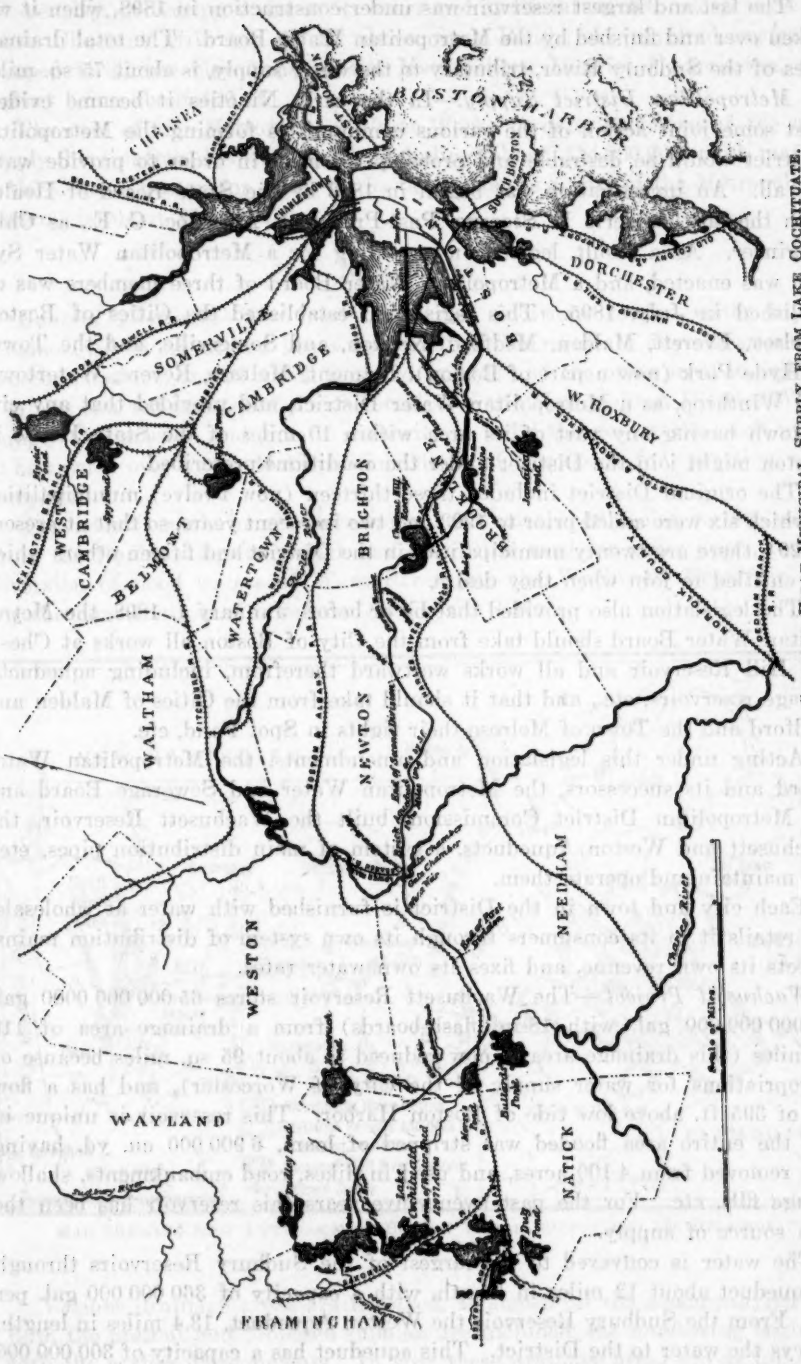
*Supplementary Supplies.*—The distribution system in the earlier days, before the introduction of high-service pumping, included small reservoirs on Beacon Hill (the site now occupied by a part of the State House in Boston) and on hills in South Boston and East Boston, all long since abandoned. Chestnut Hill Reservoir, which is still an important distribution reservoir, was finished in 1869.

A supply from Mystic Lakes was developed by the then independent City of Charlestown in 1863, became a part of the Metropolitan System in 1898, and was abandoned, because of pollution on its drainage area, when water became available from the Wachusett Reservoir.

The Cochituate Supply first threatened a shortage in 1870, and, because of the low stage of the water in the lake, pumping into the aqueduct became necessary during the following year. A temporary addition to the supply was obtained from the Sudbury River in 1871 and subsequent years, until on April 11, 1873, following a barely averted water shortage, a decision was reached by the City Council to develop a new supply from the Sudbury River, to be conveyed by a new aqueduct into the Chestnut Hill Distributing Reservoir in Boston.

*Sudbury System.*—The Sudbury Works were in the main completed, except for additional storage reservoirs on the drainage area, in 1876. At this time the total cost of the Boston Water-Works for construction, including both the Cochituate and Sudbury Systems, had been about $12 000 000.

The Sudbury Works included the Sudbury Aqueduct, with a capacity of about 109 000 000 gal. per day, 17.4 miles long, mainly a horse-shoe shaped cut-and-cover structure, 9 ft. wide by 7 ft. 8 in. high; also seven storage reservoirs varying in size from 300 000 000 gal. to 7 300 000 000 gal. and totaling about 13 000 000 000 gal.




DEVELOPMENT OF METROPOLITAN WATER DISTRICT, SHOWING FIRST MUNICIPAL SUPPLY FROM LAKE COCHITUATE.

The last and largest reservoir was under construction in 1898, when it was taken over and finished by the Metropolitan Water Board. The total drainage area of the Sudbury River, tributary to the water supply, is about 75 sq. miles.

*Metropolitan District Supply.*—In the early Nineties it became evident that some joint action of the various communities forming the Metropolitan District would be desirable and probably essential in order to provide water for all. An investigation was begun in 1893 by the State Board of Health, with the late Frederic P. Stearns, Past-President, Am. Soc. C. E., as Chief Engineer. As a result, legislation providing for a Metropolitan Water System was enacted, and a Metropolitan Water Board of three members was established in July, 1895. This legislation established the Cities of Boston, Chelsea, Everett, Malden, Medford, Newton, and Somerville, and the Towns of Hyde Park (now a part of Boston), Belmont, Melrose, Revere, Watertown, and Winthrop, as a Metropolitan Water District, and provided that any city or town having any part of its area within 10 miles of the State House in Boston might join the District under the conditions prescribed.

The original District included these thirteen (now twelve) municipalities, to which six were added prior to 1903 and two in recent years, so that at present (1929), there are twenty municipalities in the District and fifteen others which are entitled to join when they desire.

The legislation also provided that by or before January 1, 1898, the Metropolitan Water Board should take from the City of Boston all works at Chestnut Hill Reservoir and all works westward therefrom, including aqueducts, storage reservoirs, etc., and that it should take from the Cities of Malden and Medford and the Town of Melrose their rights in Spot Pond, etc.

Acting under this legislation and amendments, the Metropolitan Water Board and its successors, the Metropolitan Water and Sewerage Board and the Metropolitan District Commission, built the Wachusett Reservoir, the Wachusett and Weston Aqueducts, a system of main distribution pipes, etc., and maintain and operate them.

Each city and town in the District is furnished with water at wholesale, and retails it to its consumers through its own system of distribution mains, collects its own revenue, and fixes its own water rates.

*Wachusett Project.*—The Wachusett Reservoir stores 65 000 000 0000 gal. (67 000 000 000 gal. with 18-in. flash-boards) from a drainage area of 119 sq. miles (this drainage area is now reduced to about 95 sq. miles because of appropriations for water supply of the City of Worcester), and has a flow line of 395 ft. above low tide of Boston Harbor. This reservoir is unique in that the entire area flooded was stripped of loam, 6 900 000 cu. yd. having been removed from 4 100 acres, and used in dikes, road embankments, shallow flowage fills, etc. For the past twenty-five years this reservoir has been the main source of supply.

The water is conveyed to the largest of the Sudbury Reservoirs through an aqueduct about 12 miles in length, with a capacity of 360 000 000 gal. per day. From the Sudbury Reservoir the Weston Aqueduct, 13.4 miles in length, conveys the water to the District. This aqueduct has a capacity of 300 000 000

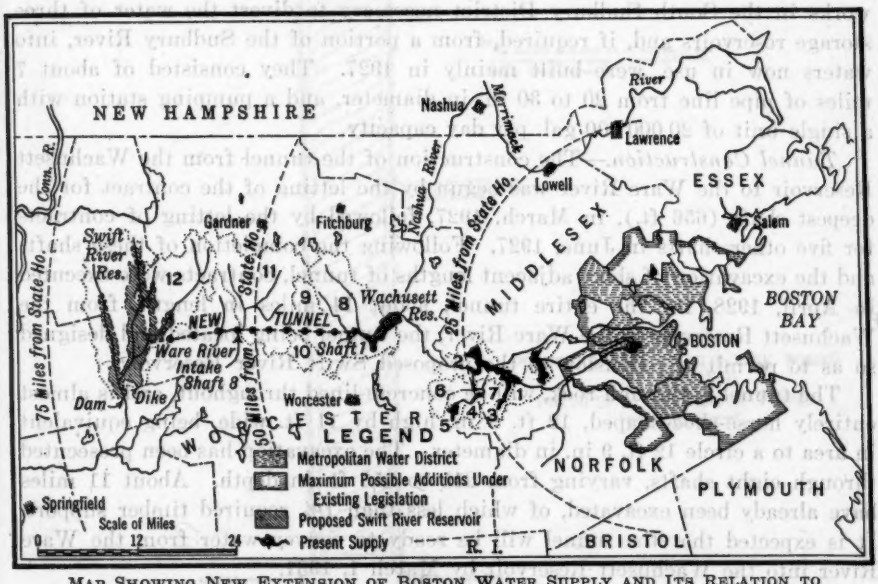
gal. per day, delivering water at its lower end at Elevation 200, which is 64 ft. above Chestnut Hill and 37 ft. above Spot Pond. As required, water is pumped to higher elevations by the Metropolitan District Commission, the highest being about 450 ft.

The cost of the Metropolitan Water-Works to date, exclusive of the new supply hereinafter described, and including about $14 000 000 paid mainly to the City of Boston for water-works taken over as a part of the Metropolitan System, is about $47 000 000.

## New Supply

*General Features.*—The need of further additions to the water supply became evident about ten years ago, and as the result of investigations by two commissions, one of which reported in 1922 and the other in 1926, legislation was enacted in 1926-27 which established the Metropolitan District Water Supply Commission and gave it authority to build the new works, such works to be turned over to the Metropolitan District Commission for operation when completed.

Provision is made for the development for emergency use of certain waters from the Sudbury System which had been little used for many years, for the diversion of flood waters of Ware River into the Wachusett Reservoir, and for the construction of a reservoir on Swift River.




Map Showing New Extension of Boston Water Supply and Its Relation to Older Systems.

Various drainage areas are involved, as shown on the accompanying map. Both the present and proposed supplies are included, the numbering beginning with the earlier developments. In size, these drainage areas are as follows:

| | Square Miles. |
|---|---|
| (1) Cochituate | 17.58 |
| (2) Sudbury Reservoir and Framingham Reservoir No. 3 | 28.22 |
| (3) Ashland Reservoir | 6.43 |
| (4) Hopkinton Reservoir | 6.26 |
| (5) Whitehall Pond | 4.35 |
| (6) Sudbury River, at Cordaville | 17.24 |
| (7) Framingham Reservoirs Nos. 1 and 2 | 12.70 |
| Total, Sudbury Drainage Area | 75.20 |
| (8) Wachusett Reservoir (except Area 9) | 95.84 |
| (9) Area allotted to Worcester | 13.00 |
| Total present Wachusett Drainage Area | 108.84 |
| (10) Area diverted by Worcester | 9.35 |
| Total original Wachusett Drainage Area | 118.19 |
| (11) Ware River Intake | 98.00 |
| (12 Proposed Swift River Reservoir | 186.00 |

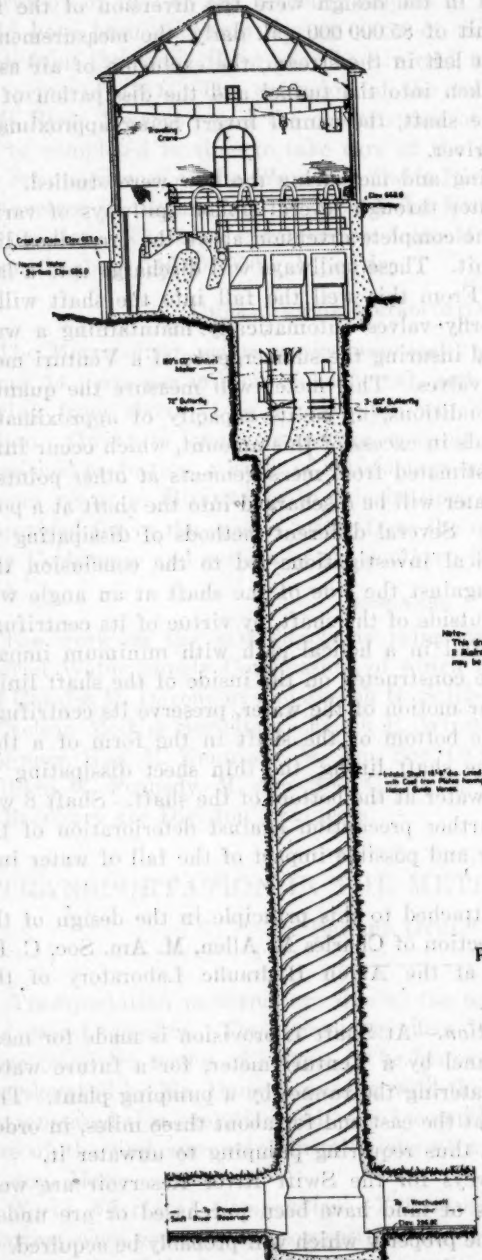
*Route of Water.*—The water from Ware and Swift Rivers will be conveyed to the Wachusett Reservoir through a tunnel about 25 miles in length. The flow line of the Wachusett Reservoir, when full, is 260 ft. lower than the Ware River Intake, and fully 120 ft. lower than the proposed Swift River Reservoir. As the Ware River will enter the tunnel at about midway of its length, through a shaft about 260 ft. deep, the flood waters may be stored in either the Wachusett or the Swift River Reservoir, as desired.

The organization of the Engineer Corps began in October, 1926. The works in the South Sudbury District necessary to divert the water of three storage reservoirs and, if required, from a portion of the Sudbury River, into waters now in use, were built mainly in 1927. They consisted of about 7 miles of pipe line from 20 to 30 in. in diameter, and a pumping station with a single unit of 20 000 000 gal. per day capacity.

*Tunnel Construction.*—The construction of the tunnel from the Wachusett Reservoir to the Ware River was begun by the letting of the contract for the deepest shaft (656 ft.), in March, 1927, followed by the letting of contracts for five other shafts in June, 1927. Following the completion of these shafts and the excavation of short adjacent lengths of tunnel, contracts were executed in April, 1928, for the entire tunnel, about 14 miles in length, from the Wachusett Reservoir to the Ware River, the tunnel being located and designed so as to permit of extension to the proposed Swift River Reservoir.

The tunnel is through rock, will be concrete-lined throughout, and is almost entirely horse-shoe shaped, 12 ft. 9 in. high by 11 ft. wide, being equivalent in area to a circle 12 ft. 9 in. in diameter. The excavation has been prosecuted through eight shafts, varying from 215 to 656 ft. in depth. About 11 miles have already been excavated, of which less than 1% required timber support. It is expected that the tunnel will be ready to convey water from the Ware River into the Wachusett Reservoir by March 1, 1931.

Ledge rock at the shafts was found within 5 to 30 ft., except in one instance where the earth cover was about 60 ft. The rock is generally sound, of schistose character, with extensive granitic intrusions. The tunnel excavation as a whole is comparatively dry, the water coming in mainly along joints and crush zones, but not in large quantities.




VERTICAL INTAKE SHAFT, SHOWING SIPHON INLET AND SPIRAL LINING.

*Ware River Diversion Works.*—The Ware River Intake to the tunnel will consist of Shaft 8 and the diversion dam and control works at the surface. Four principal considerations in the design were the diversion of the flood flows above the prescribed limit of 85 000 000 gal. daily, the measurement of the quantity diverted and that left in the stream, the exclusion of air as far as possible from the water taken into the tunnel and the dissipation of the energy due to the drop in the shaft, the tunnel invert being approximately 260 ft. below the level of the river.

Various methods of diverting and measuring the flow were studied. The general plan is to take the water through several siphon spillways of various capacities, which will insure the complete diversion above the prescribed limit and prevent any below this limit. These spillways will discharge into a large well directly over the shaft. From this well the fall into the shaft will be controlled by four large butterfly valves automatically maintaining a water seal for the exclusion of air and insuring the submergence of a Venturi meter set vertically over one of the valves. This meter will measure the quantity diverted under all ordinary conditions, up to its capacity of approximately 700 000 000 gal. daily. For floods in excess of this amount, which occur infrequently, the quantity will be estimated from measurements at other points.

*Spiral Water Flow.*—The water will be discharged into the shaft at a point about 260 ft. above the tunnel. Several different methods of dissipating the energy were studied. Theoretical investigations led to the conclusion that water discharged tangentially against the side of the shaft at an angle with the vertical would keep to the outside of the shaft, by virtue of its centrifugal force, and maintain a smooth fall in a helical path with minimum impact. Also, that if helical guides were constructed on the inside of the shaft lining they would maintain the circular motion of the water, preserve its centrifugal force, and cause it to reach the bottom of the shaft in the form of a thin sheet pressing firmly against the shaft lining, the thin sheet dissipating its energy in an adequate depth of water at the bottom of the shaft. Shaft 8 will be lined with cast iron as a further precaution against deterioration of the concrete under the high velocity and possible impact of the fall of water into the shaft.

Because of the importance attached to this principle in the design of the shaft lining, tests, under the direction of Charles M. Allen, M. Am. Soc. C. E., were made on a $\frac{1}{20}$-scale model at the Alden Hydraulic Laboratory of the Worcester Polytechnic Institute.

*Wachusett Reservoir Connection.*—At Shaft 1, provision is made for measuring the flow through the tunnel by a Venturi meter, for a future water power development, and for unwatering the tunnel by a pumping plant. The grade of the tunnel was lowered at the east end for about three miles, in order to ensure support in sound rock, thus requiring pumping to unwater it.

*Swift River Reservoir.*—Surveys for the Swift River Reservoir are well advanced and about 35 000 acres of land have been purchased or are under option, this being about 60% of the property which will probably be acquired.

This reservoir will require two dams of about the same dimensions. The maximum heights will be about 120 and 150 ft. above the earth, and about 250 ft. above the rock. The capacity of the reservoir will probably exceed 300 000 000 000 gal.

*Dates Required for Completion.*—The safe capacity of present sources of water of good quality has already been reached, and but for the heavy rainfall and run-off of 1927-28 and the early months of 1929, the use of inferior water would have been necessary during parts of this period. As already stated, water from the Ware River may be diverted to the Wachusett Reservoir in the spring of 1931. The extension of the tunnel westerly to the proposed Swift River Reservoir and the construction of this reservoir, it is expected, will be completed in time to take care of the growing needs of the District. The estimated cost of the Ware and Swift River projects is $65 000 000, and the increase in the available supply of the District will be about 191 000 000 gal. per day. The total consumption from Metropolitan District sources at present is about 140 000 000 gal. per day.

### Character of Metropolitan Water

The Metropolitan water carries practically no turbidity, is uniformly soft (about 14 parts per million of hardness), and the color is not objectionable, varying from about 14 to a maximum which seldom exceeds 20 parts per million. Owing to long-time storage, sparsely populated drainage areas, and water-shed protection, a water of satisfactory sanitary quality has been secured without resort to filtration or to chlorination, except that the use of chlorine is provided for in the case of water from the more polluted drainage areas of Lake Cochituate and certain of the southern Sudbury sources.

### Personnel

The work on the new supply is being carried on by the Metropolitan District Water Supply Commission, of which Davis B. Keniston is Chairman and Charles M. Davenport and Joseph H. Soliday are Associate Commissioners. R. Nelson Molt is Secretary. Frank E. Winsor, M. Am. Soc. C. E., is Chief Engineer, Karl R. Kennison, M. Am. Soc. C. E., is Designing Engineer, and William W. Peabody, M. Am. Soc. C. E., N. LeRoy Hammond, and Richard R. Bradbury are Division Engineers.

## TRANSPORTATION IN THE METROPOLITAN DISTRICT

By Edward Dana,* Esq.

Transportation problems are due to the tendency of people to congregate and crowd together. The increasing effect has been to displace earlier and more primitive modes of transit. The history of transportation development was sketched by Mr. Dana beginning with the period just before the Eighties. He remarked that evolution had been accomplished roughly in 10-year periods. One of the first successful installations of an electric railway was made in Boston, Mass.

*Early Effects of Competition.*—Following this installation a number of charters were granted and competition was so keen that sometimes the light cars were derailed, circled around a competitive line, and hurried back to their origin in order to get business.

* Gen. Mgr., Boston Elev. Ry., Boston, Mass.

"The result was that such competition was not good for the community," said Mr. Dana. "It was realized that it was wasteful, that it was a 'dog-eat-dog' kind of service, and that if real service was to be rendered there should be a unification and consolidation."

*Consolidations.*—The result was the formation of the old West End Street Railway; but this did not entirely solve the problem because the electric car was not yet fast enough. Steps were then taken to provide better service and that was the beginning of another 10-year epoch. Mr. Dana describes this period as follows:

"In 1898 the Boston Elevated Railway was chartered for the purpose of leasing the West End Street Railway and providing, out of capital raised by stocks and bonds, for some real rapid transit. Congestion was quite a problem even then, before we heard of the automobile. It had been heard of, but they were few and far between on the street. Congestion occurred * * * and it was necessary, if business was to prosper and if the area was to grow so that we would have a large metropolitan city, to have some kind of rapid transit. The Boston Elevated was chartered and took over the franchises of the West End. The investment at that time was $25 000 000. Boston blazed the trail—with civil engineers assisting—in building the first subway on this continent for electric cars, the present Tremont Street Subway."

The result was that new areas were developed and the city began to grow rapidly until about 1908 when there was a cry for further developments.

*Early Difficulties.*—Progress, however, was arrested in the next ten years which was the logical time for action. The city was operating on a flat 5-cent fare, "which was harsh because it was determining the price of a commodity without any regard for its characteristics as a commodity and without regard to cost of production."

In 1912, a long-deferred adjustment of wages was made, which was rapidly increased later when the United States entered the World War. In 1918 when the War Labor Board fixed wages to be effective on electric railways, it brought the Boston situation to a crisis. This was met by running the Boston Elevated on a service-at-cost basis under the control of a Board appointed by the Governor.

*Provision for Depreciation.*—Commenting on one phase of this period, Mr. Dana remarked:

"In 1918, on taking over the reins of this high-powered machine, the Board did not fix the price of the commodity quickly enough to prevent calling into effect the safety valve which was put into the law; namely, the assessment of the cities and towns in case of a deficit. It was unfortunate that that had to be, because the impression has gone abroad that it is very simple to operate the Boston Elevated; all you have to do is to call upon the taxpayers and they will pay the deficit.

"An effort has been put forth since 1918 to operate the system efficiently and economically within the limits of a 9.2-cent fare. Boston is not a straight 10-cent fare city. * * * About one-fifth of the riders are carried on short-haul, 6.25-cent tickets.

"During those ten years there has been spent, out of depreciation (which was a requirement under the Act) certain moneys in order to make possible the efficient functioning of the system. That depreciation is very much misunderstood. It is simply the replacement of worn-out capital through public control. There is nothing to make up for back depreciation prior to 1918, but the life of every individual unit of the property is taken, and an estimate

made of the life it should have, and the proper amount charged in the service at cost. It is true that we have been able to put in new power station units and some modern cars that compare favorably with, if they are not better than, those put in elsewhere, and we have gotten certain economies."

*Plant and Personnel.*—For example, the Trustees have spent $4 500 000 for certain improvements that resulted in such marked efficiency and economy that the payroll is now about $500 000 less than it was eleven years ago (1918). The original investment, $25 000 000 in 1898, is now (1929) $176 000 000, and the per capita investment is greater than in any city in the United States. Detroit, Mich., for example, has a per capita investment of $23 as compared with $150 in Boston.

The Boston Rapid Transit System is in a position to serve 91 sq. miles of territory and a population of 1 300 000. Its employees number 8 000 as compared with 10 500 a few years ago, and Mr. Dana acknowledged the loyalty of the workers in helping to create the improvements thus far made.

*Peak Loads of Automobile Users.*—He alluded to the effect of the automobile in changing the character of service required of a rapid transit system. In his own words:

"There has been a change in the trend of riding, and we lose one-third of the business in July and August. A loss of one-third of the business for three months eats into the total amount of revenue for twelve months. It means quite a reduction in the figures of the number of people served, and there are many worthy gentlemen in the community who try to point out that the Elevated is not needed, because it is not carrying so many people as it carried five or ten years ago. We found out that our power load (which is a pretty good index) and our actual operating figures show, that on week days, eight months of the year, there is an increase. Each winter we have been having a harder problem in carrying more people. It means there is a differentiation of service with that fixed charge of $176 000 000 going on all through the year. It means that there is a very critical problem there eight months of the year, and that problem, as I look upon it, has a potential danger in addition to the swing from summer to winter. * * * If we get a northeast snowstorm in December and another one closely following, we have a potential load of 45 000 to 55 000 people who use their automobiles through the rest of the year.

"If capital were to be available to carry a reserve power plant, a reserve in cars and men, the increase would be considerable. Under labor agreements it is quite a problem to be able to put your hands on enough men and to carry them on a guaranteed eight-hour day throughout the year. It is quite a problem to have to face in three months a potential load, varying according to the severity of the storm, of 45 000 to 55 000 people a day. The only answer is to recognize the need for more rapid transit."

## SEWERAGE IN THE BOSTON METROPOLITAN DISTRICT

By X. Henry Goodnough,* M. Am. Soc. C. E.

In the absence of Mr. Goodnough, his paper was delivered in condensed form by F. H. Kingsbury, M. Am. Soc. C. E., Assistant Engineer of the Massachusetts Department of Public Health. In common with the other papers deal-

* Chf. Engr., Massachusetts State Dept. of Health, Boston, Mass.

ing with the Boston Metropolitan District, Mr. Goodnough's description reviewed the historical bases for the development of sewerage systems.

*Early Conditions.*—One of the chief early factors in this situation was the geographical location of Boston—a series of peninsulas with three tidal rivers emptying into Massachusetts Bay, and a tidal range amounting to 12 to 15 ft. at its maximum. Under these conditions the sewers were not all that could be desired, "but on the whole probably served their purpose fairly well, chiefly because the old city was built for the most part on hills with steep slopes to the water on all sides."

As far back as 1850 there appears to have been 35 miles of underground sewers in the city; by 1869, this had increased to 100 miles; in 1885, to about 225 miles; and, at present, the total amounts to approximately 1 000 miles. This is exclusive of intercepting sewers.

With the growth of the city, it became necessary to fill in many of the surrounding tidal flats for purposes of legitimate city use. This entailed the extension of the sewer line, necessarily at very flat grades, with the inevitable result of creating "offensive odors caused by the deposits of sewage in the neighborhood of the outlet." In consequence, a Commission was appointed by the City Government, and a detailed report was made in 1876. Certain of its recommendations were followed, especially as related to the part of the city south of the Charles River. The territory to the north did not participate in the sanitary reconstruction recommended. These plans were the basis for the so-called "Boston Main Drainage System."

## Main Drainage System

*Design.*—Although the total area comprised within the theoretical drainage system amounted to about 58 sq. miles, it appeared feasible at the time to take care of only approximately 20 sq. miles lying near the shore.

The plan "provided for the construction of intercepting sewers around the east and west sides of the main portion of the city and along the north and south shores of South Boston," also a sewer from the direction of Roxbury. These various lines were led to a pumping station where the sewage was "raised and delivered through a tunnel under Dorchester Bay into reservoirs at Moon Island, whence it was discharged into the sea on the outgoing tide." The general layout of this system is shown on the accompanying map.

Many interesting facts relative to the design of this system were cited by Mr. Goodnough. For example, the sewers were designed to flow half full, generally on a slope of 1 in 2 500. Only a pre-determined amount of flow could enter the intercepting sewers, and the remainder in time of storm would overflow directly to tide water. Means to prevent the tide water from backing up in the sanitary sewers and other safeguards were also enumerated by Mr. Goodnough.

*Disposal.*—For the disposal of the sewage, comprehensive works were provided, which Mr. Goodnough described in some detail. Looking back, with the advantage of modern development in this field, it is of interest to note that,

> "The pumping and outfall works have been changed but little since their construction nearly fifty years ago, except for the number and type of pumps in use."

Finally, the sewage reaches Moon Island, about 1 mile offshore from the mainland at Squantum, where reservoirs have been provided, having at present a capacity of 50 000 000 gal. In principle, the operation here is quite simple:

"The sewage accumulates in the reservoirs during the time of the incoming tide and is discharged in the second and third hours of the outgoing tide, except possibly at times of very heavy storms, when the discharge is begun earlier or continues for a longer period."

According to statistics quoted by Mr. Goodnough, the quantity pumped, in terms of average daily quantity, has increased more or less gradually. In 1885, it amounted to about 34 000 000 gal. per day; in 1900, to about 75 000 000 gal.; in 1915, to about 101 000 000 gal.; and in 1927 (the peak), to 124 000 000 gal. daily.

*Operation.*—Reviewing the experience of the City with this system, Mr. Goodnough remarked that,

"In the construction of the Moon Island System the first and most essential step was taken in providing sewerage for the Boston Metropolitan District. On the whole, the works have operated very satisfactorily during the forty-five years that they have been in use. No material changes have been made in the main system."

## North Metropolitan Sewerage System

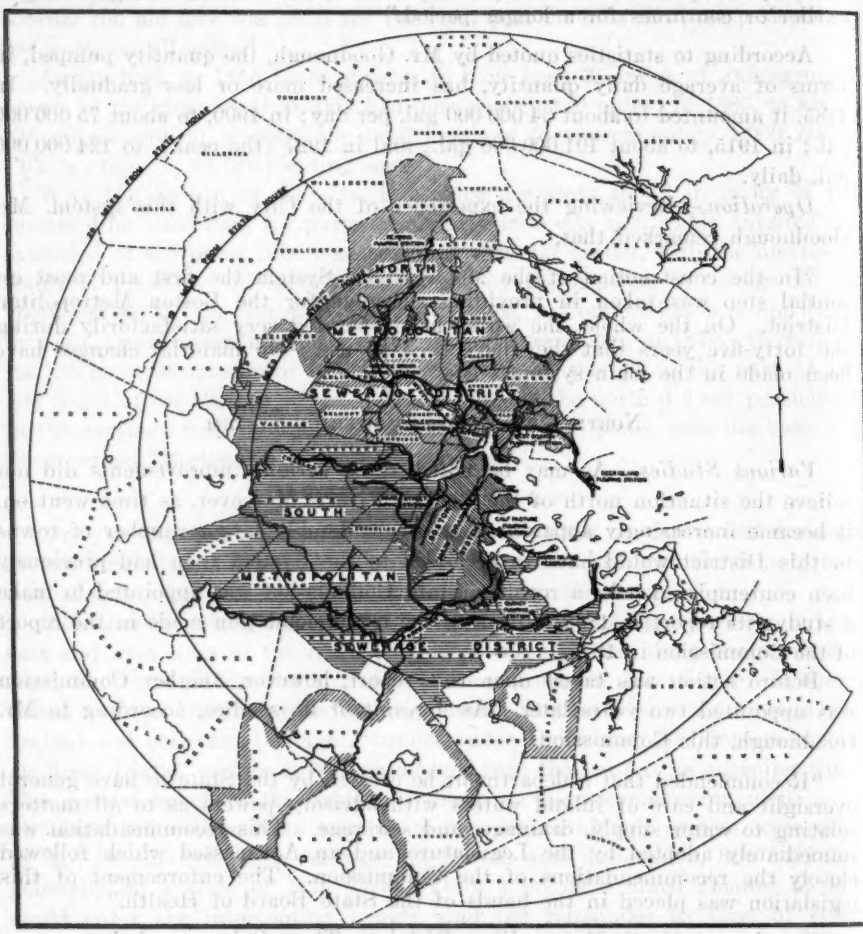
*Various Studies.*—As may be noted, these various improvements did not relieve the situation north of the Charles River. Moreover, as time went on, it became increasingly apparent that a considerably larger number of towns in this District would have to be added to the System than had previously been contemplated. As a result a State Commission was appointed to make a study; its report in 1882 reinforced the recommendation made in the report of the Commission in 1875.

Before action was taken upon this report, however, another Commission was appointed two years later. As a result of its studies, according to Mr. Goodnough, this Commission.

"Recommended that a department be created by the State to have general oversight and care of inland waters with advisory powers as to all matters relating to water supply, drainage, and sewerage. This recommendation was immediately adopted by the Legislature and an Act passed which followed closely the recommendations of the Commission. The enforcement of this legislation was placed in the hands of the State Board of Health."

*Creation of North Metropolitan District.*—Thus, it happened that an entirely new State body came to have an important influence on the choice of sewerage works for the district north of the Charles River. It took several years for its investigations and reported finally to the Legislature in 1889. Its careful studies led to the rejection of the methods of treatment on land previously recommended, because of, "the impracticability of disposing of such a large quantity of sewage upon the area recommended by any practicable plan of treatment then known."

"As a result of its studies the State Board of Health recommended the creation of a North Metropolitan sewerage system with an outlet into the sea near Deer Island Light and a Charles River system to connect with the existing main drainage system of the City of Boston. The report of the Board was accepted by the Legislature and a commission known as the Metropolitan Sewerage Commission was appointed in that year and directed to construct the works substantially in accordance with the plans recommended by the State Board of Health."




MAIN SEWERS IN THE BOSTON METROPOLITAN DISTRICT.

*General Plan.*—This system became known as the North Metropolitan Sewerage System, and the works were completed in 1894. The main sewers are shown on the accompanying plan. As described by Mr. Goodnough,

"The main line in general follows the northerly bank of the Mystic River through Medford, Everett, and Chelsea to East Boston, where the sewage is raised about 15 ft. by pumping, whence it flows through East Boston and Winthrop to a second pumping station at Deer Island, at which it is raised 11 ft. and discharged into deep water off Deer Island Light."

Drainage from some of the lower areas concerned was introduced into this main system by pumping at several locations:

"Some of these trade wastes, notably those from tanneries, are passed through settling tanks for the removal of heavier matters before they are admitted to the sewers. Aside from this limited sedimentation process, the sewage receives no treatment before its final discharge except that afforded by coarse screens at the pumping stations designed to prevent large objects from reaching the pumps."

In general, the system was planned for a flow of 225 gal. per capita per day. In some of the communities, the sewerage systems were on the combined plan, and for these an additional allowance of about 40 gal. per capita per day was made. At first, the sewage was discharged directly into the ocean about 1 900 ft. out from the Deer Island shore, in a depth of about 30 ft. of water at low tide. In 1915, however, an extension was built about 300 ft. farther out; this was provided with a number of openings for the purpose of distributing the effluent over a larger area and thus securing greater dilution.

*Operation.*—The simplicity of operation is thus described by Mr. Goodnough:

"The sewage is discharged as rapidly as it is received at the pumping station continuously at all stages of the tide. On the incoming tide the sewage flows toward the northerly part of the harbor and is traceable for a distance of ½ to ¾ mile from the outlet. The area affected by the sewage, so far as bacterial analyses show, is usually about 300 acres, although the sleek is observable over a wider area. At the turn of the tide the sewage flows directly to sea covering a smaller area than on the incoming tide, and is quickly dispersed in the water.

"The slight effect of this sewage upon the waters of the harbor and the smaller area affected by it as compared with the discharge at Moon Island is due no doubt in part to the depth at which the sewage is discharged and in part to the much greater velocity of the current which passes Deer Island Light."

In spite of the very satisfactory operation of the North Metropolitan Sewerage System, the inevitable growth forecasts the necessity—almost the immediate necessity—for a "further provision for the collection and disposal of the sewage of this area."

## South Metropolitan Sewerage System

Subsequent to the construction of the Main Drainage System, various additions were made to the original Southern System. Among these was a main sewer through Neponset Valley, completed in 1897. This brought drainage from some of the outlying sections toward the south, and discharged it into the Boston Sewer System.

*High-Level Sewer.*—A high-level sewer in the Southern District was projected as far back as 1875. With the growth of this section the Main Drainage Works became more and more inadequate; in 1896 this system was taking care of about 120 sq. miles or, approximately, twice that for which it was designed. As a result the Metropolitan Sewerage Commission was directed" to prepare plans for a high-level gravity sewer to relieve the district south of the Charles River."

In brief, its proposal was to build on independent system on a higher level to take the discharge from a large part of the area and lead it directly to an outlet in tide water at Peddocks Island. The State Board of Health, to which these plans were referred, approved them practically in their entirety, and, accordingly, the system was built.

As summarized by Mr. Goodnough,

"This system was designed to receive the sewage from the Charles River and Neponset River valleys, from the high lands of Dorchester and West Roxbury, and from the City of Quincy."

Starting at a level of about 31 ft. above low water it reaches a level of about 10 ft. above low water near the shore, discharging through three main outlets of 5-ft., cast-iron pipes, 1 500 ft. apart, near Nut Island (see the map).

Some of the sewage from the City of Boston is pumped up into this high-level sewer, while that from the City of Quincy is partly intercepted by gravity or partly pumped. Since its completion, several additions to this system have been made, notably that in 1919, to serve the Towns of Needham, Wellesley, etc. The present area of the District is about 190 sq. miles.

*Disposal of the Sewage.*—As Mr. Goodnough describes the sewage treatment:

"The sewage of the South Metropolitan District is discharged continuously into the sea as it reaches the outlet at all stages of the tide. No storage is provided at any point and the sewage receives no treatment other than coarse screening at the pumping stations and at the screen house at Nut Island. * * * No fouling of the shores has taken place in the neighborhood of the outlets, and there has been no complaint of objectionable conditions in the harbor traceable to the sewage discharged at this outlet. The district served by the South Metropolitan System is largely suburban, although it contains several important industrial areas. Industrial wastes in the district, however, have thus far been admitted to the sewers without preliminary treatment."

## Statistics

As to the comparable quantities of sewage handled since the North and South Districts have been in operation, Mr. Goodnough quoted the following figures, in terms of gallons per day:

| Year. | North Metropolitan Sewerage District. | | South Metropolitan Sewerage District. | |
|---|---|---|---|---|
| | Average. | Maximum. | Average. | Maximum. |
| 1905 | 54 400 000 | 126 600 000 | 25 000 000 | 78 000 000 |
| 1910 | 59 000 000 | 135 100 000 | 39 600 000 | 141 000 000 |
| 1915 | 60 392 000 | 151 600 000 | 52 300 000 | 155 000 000 |
| 1920 | 74 000 000 | 151 200 000 | 71 700 000 | 234 000 000 |
| 1925 | 78 100 000 | 148 000 000 | 63 700 000 | 166 000 000 |
| 1928 | 86 900 000 | 150 700 000 | 64 900 000 | 162 000 000 |

In passing, he notes that the original design of the North Metropolitan System provides for about 135 000 000 gal. per day and the South Metropolitan System, 300 000 000 gal. per day. The growth in population tributary to the various parts of the System is graphically illustrated in the chart herewith.

The costs of operation are also given, as follows:

| Year. | North Metropolitan Sewerage System. | South Metropolitan Sewerage System. |
|---|---|---|
| 1910 | $141 318.37 | $ 93 421.13 |
| 1911 | 150 137.69 | 100 841.40 |
| 1912 | 156 221.55 | 105 403.52 |
| 1913 | 168 790.63 | 104 968.59 |
| 1914 | 209 674.99 | 105 958.35 |
| Average: 1910-1914, inclusive..... | $165 228.65 | $102 118.60 |
| 1924 | $316 723.83 | $217 184.42 |
| 1925 | 329 833.72 | 198 789.20 |
| 1926 | 322 285.18 | 198 025.99 |
| 1927 | 331 228.03 | 204 118.03 |
| 1928 | 352 280.07 | 235 768.01 |
| Average: 1924-1928, inclusive..... | $330 470.17 | $210 777.13 |
| Percentage of pre-war average. | 200 | 206 |

The comparison between maintenance costs before the World War and since are quite marked.

To September 1, 1929, the total capital costs of various parts of the Systems have been approximately:

| | |
|---|---|
| North Metropolitan System..... | $ 8 700 000 |
| South Metropolitan System..... | 10 000 000 |
| Total ..... | $18 700 000 |

*Assessments of Costs.*—In consequence of the number of various cities and towns enjoying the benefits of the comprehensive sewage works, the division of costs becomes a rather complicated matter. Hence, for the North Metropolitan District, three commissioners were appointed by the Supreme Judicial Court, whose report in 1891 concluded that,

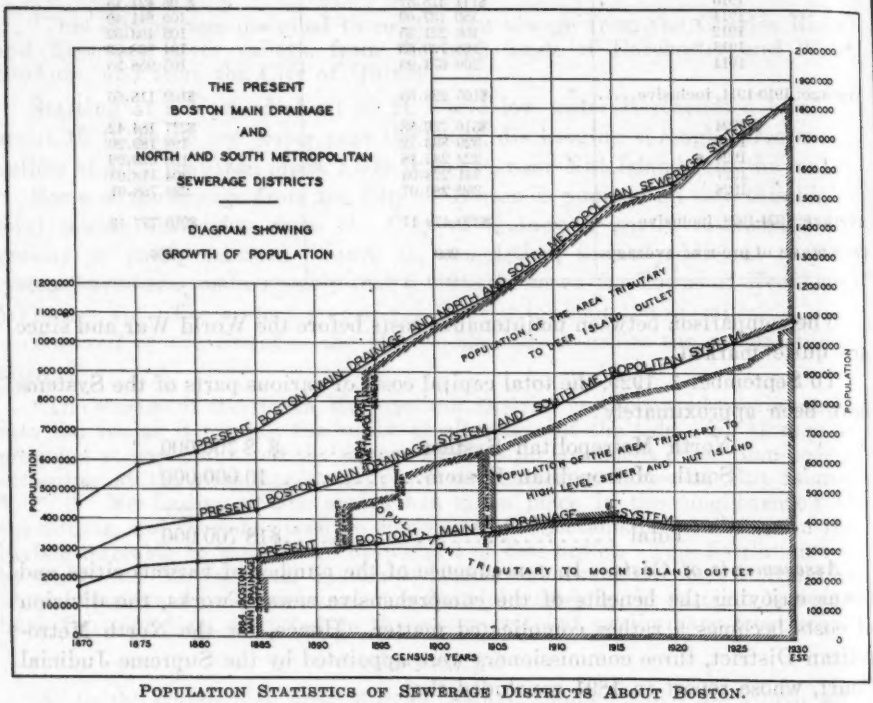
"There is no method more just and equitable than to apportion the payments on account of the interest and sinking fund requirements of the loan, which represents the first cost of the systems, with primary reference to the total taxable valuations of the several cities and towns; and the payments on account of the cost of maintenance and operation, with primary reference to their respective populations;* * *."

Variations from these apportionments were provided, for cases of extreme conditions. In these determinations, the valuations of May 1, 1890, and the population according to the National census of that year were adopted as the bases of apportionment.

So satisfactory was this system that it continued in operation until a change was made by a legislative act in 1906; this, stated Mr. Goodnough, provided that

"Future division of costs should be made by the then Metropolitan Water and Sewerage Board by dividing the cost of payments for interest and sinking fund among the cities and towns upon the basis of their respective valuations and the cost of maintenance and operation upon the ratio of the respective populations of the several towns to the total. This method of apportionment still remains in force.

"When other cities and towns are admitted to the existing districts their assessments are computed in the same manner as the others, but upon the admission of a city or town to a metropolitan sewerage district the municipality is required to pay an admission fee which usually represents its proportionate share of the sinking fund in the ratio of its valuation to the total valuation of the district."




POPULATION STATISTICS OF SEWERAGE DISTRICTS ABOUT BOSTON.

## SUMMARY

Giving a bird's-eye view of the extent of all this work, Mr. Goodnough remarked that,

"The Metropolitan Sewerage and Boston Main Drainage Districts include at the present time 32 cities and towns having an aggregate area, including parks and other public and quasi-public reservations, of 304 sq. miles. Of this 44 sq. miles are comprised within the limits of the City of Boston, and the remaining 260 sq. miles include the 31 cities and towns in the Metropolitan Districts outside the city.

* * * * * * *

"Compared with the population 50 years ago when construction of the Boston Main Drainage System was first begun, the population of the City of Boston has increased about 230%, while that of the Metropolitan District outside the limits of Boston has increased nearly 400 per cent."

As to present conditions and future prospects, he stated that,

"The experience of half a century of the operation of the Boston Main Drainage System has shown it to be admirably adapted to the purpose which it was designed to serve and testifies to the thoroughness with which the

problem was studied by the engineers to whom the work was committed. The construction of the works was a training school for engineers and supplied practical experience on which the design of the later works was largely based. It still serves adequately the population in the area which it was ultimately designed to serve and no changes of consequence have been required since its completion.

"The North and South Metropolitan Systems have also served equally well the purpose for which they were designed and little change has been required in either of them. * * * Industrial wastes have not affected materially the sewerage systems except that gasoline has been excluded from the sewers on account of the danger of the creation of explosive mixtures.

"With the approach of the end of the period for which the North Metropolitan System and the Charles and Neponset Sections of the South Metropolitan System were designed to serve, it is probable that some additional provision will have to be made for the collection and removal of the sewage from parts at least of those areas; and, in fact, the Legislature has already ordered an investigation of the various sewer outlets and the effect of the sewage upon Boston Harbor to determine what changes or enlargements, if any, may be necessary in the existing systems. This investigation is now in progress.*

"Until the results are available it is impracticable to predict what changes will be found desirable in the present method of sewage disposal in this Metropolitan Area. Whatever process of treatment, if any, may be selected or whatever change may be made in the points of discharge of the sewage, the District will be fortunate if the results prove to be as satisfactory as the methods adopted by the engineers who designed the Boston Main Drainage and the Metropolitan Sewerage Systems thirty years and more ago."

## GENERAL DISCUSSION

*Legislative Needs.*—The comparison between conditions in Massachusetts and New Jersey, with special reference to the Oranges, was elaborated by Alexander Potter, Assoc. M. Am. Soc. C. E., who stated:

"At the present time it is utterly impossible, in the State of New Jersey to form districts. Consequently some of the sewerage plans and water supply plans cannot be constructed for the benefit of all when they require the voluntary action of the towns both as to the size of the sewer, capacity, etc., that they feel constrained to pay for. You can readily understand how difficult it is to get eleven or thirteen towns (as is the case in one project) together and have them make a common decision. Furthermore, the laws are such that we do not get the co-operation as freely as we might. All of the projects except one, I think, are done by the voluntary act of the towns together. Even at the present time some towns have greater capacity than others, and to dispose of that capacity through their towns it has to be by consent of the other municipalities. If that can be solved the question of these public improvements can be solved in New Jersey as well as in other States."

*The Water Supply.*—Some discussion from the floor was directed to the paper on water supply. Replying to a question by Kenneth Allen, M. Am. Soc. C. E., Mr. Winsor explained that the tunnel from the Swift River is fixed by the hydraulic line between the Swift River and the Wachusett Reservoirs. It is fixed by the line from the surface of the water at the Ware

* Chapter 29 of the Resolves of 1929.

River to that at the Swift River. The sole purpose of lowering the tunnel at the east end was to get good solid rock. The change in gradients was 2.5 to 3 miles adjacent to the Wachusett Reservoir. Paul N. Ivancich, Jun. Am. Soc. C. E., called attention to certain legal problems sometimes encountered on highway construction in locating a route through graveyards. Similar problems, according to Mr. Winsor, are being taken care of on the Boston Water Supply project at the cost of about $5 000.

The question was raised by Eugene D. Barstow, M. Am. Soc. C. E., as to whether communities within the Metropolitan Area could exercise their own option in the matter of using the Metropolitan Water Supply and Sewerage Systems. Mr. Winsor replied that:

"The original legislation in 1885 which created the Metropolitan District of thirteen cities and towns, provided that any other city or town within ten miles of the State House might come in at any time they desired. When this legislation was passed all these cities and towns had some water supply, but many of them were on the 'ragged edge' and had to have more right away. Those that had were put in the act as constituting the Metropolitan Water Supply Area; but before the water became available I think there were thirteen cities and towns in the Metropolitan District, and within the last few years two or more have come in, making fifteen.

"According to Mr. Keniston, the original act required certain cities and towns to be put into different systems, but I think it was more or less voluntary on the part of most of the municipalities; perhaps some were forced to go in. Since that time additions have been made from time to time by special act, and I think that in most cases it was largely upon the request of the municipality itself by filing a bill in the Legislature and getting it enacted. There is a general provision by which cities and towns can join the Metropolitan System without special legislation."

*Transportation.*—In discussing the transportation problems, Edwin F. Wendt, M. Am. Soc. C. E., brought out that the depreciation reserve, mentioned by Mr. Dana, may be either a depreciation fund or simply a depreciation allowance. He stated his understanding that Mr. Dana's reference

"Was merely a bookkeeping allowance in order that the car rider may pay a sufficient amount to preserve the property in good operating condition and not, in fact, a real genuine depreciation reserve represented by a fund in bank."

# CITY PLANNING DIVISION

OCTOBER 10, 1929—10:30 A. M. TO 12:30 P. M.

## THE PLAN OF BOSTON: A CAPITAL CITY

By Arthur C. Comey,* M. Am. Soc. C. E.

*The Largest Capital.*—The chief justification of any description of a capital city, according to Mr. Comey, "lies in the inspiration and guidance which the capital city may exert through its example on other cities within the State." As an illustration the capital of the United States, itself well-planned, has a great effect on the other large cities of the Nation. In the same way, but on a smaller scale, State capitals may reign within the sphere of State influence.

In many capitals, especially those of small size, the State's functions may predominate over all other city activities. The other extreme, said Mr. Comey, is illustrated by the City of Boston,

"The largest of all our capitals, in which the presence of the Capitol is but an incident—the one large Capitol Building, the crowning jewel of an architectural composition in one portion of the city, to be sure, but otherwise exercising practically no effect upon the city plan.

"Boston's plan and Boston's effect upon the surrounding region are due to its size, to its being the commercial center of this region, to its being a seaport, and to the industrial, educational, and other activities that have been attracted to it."

*Historical Background.*—Furthermore, Boston in its development has been largely governed by the historical nature of its layout,

"Never planned in any large degree, it has grown steadily from a small Seventeenth Century Colonial village and carries in its layout many visible evidences of each successive era in its practically three hundred years of life."

To illustrate this influence, Mr. Comey described the adjustments made on the original site. Many of the narrow, crooked streets still remain, also, fortunately, "many architectural gems, both public and private." As the surrounding mud-flats were filled, however, the peninsula lost its distinctive shape, and large areas were made available for new planning. In general, these new developments were laid out on the rectangular checker-board system.

Then, too, outlying districts were added from time to time and impressed their own layouts on the general plan. This resulted in many inconsistencies, especially on through streets, where, for example, a highway such as Washington Street may be intermittently 50 or 100 ft. wide, without logic as to traffic requirements.

As a result of these and other misfits, Mr. Comey describes the outlying sections as "for the most part a hodge-podge of unrelated fragments, narrow

* Landscape Archt. and Cons. City Planning Engr., Cambridge, Mass.

streets leading nowhere." Fortunately, there are exceptions to this condition, and in the outer sections where the land was cheap the through highways were widened to good advantage.

*Planning Commissions.*—One of the great constructive influences within the last forty years has been the development of a park system, as described by another paper.* A further effective influence has been the establishment of a "Board of Survey", which has attempted to unify the general scheme of wider streets. An effort to invoke the police power to establish this plan was not a success, but still a great many of the Board's recommendations were followed voluntarily.

Various attempts to widen streets have been successful, notably that following the fire of 1872. In fact, Mr. Comey stated,

"It was estimated a number of years ago that $40 000 000 had then been spent on widening and straightening, but 'not all of Boston's streets are wide or straight yet.' At first, these improvements were successive, isolated projects. Several streets have been widened two or even three times, while other proposals have languished for many years before being carried out."

Finally, in 1914, a permanent official Planning Board was established with advisory powers only. To its credit are many needed improvements, said Mr. Comey, for example:

"It successfully launched a general zoning scheme in 1924 and since then has accumulated data, and, with the aid of a special staff, is now about to publish a comprehensive major street plan relating all street improvements to one another and to the city's ultimate needs."

*Water Development.*—Boston's drainage problems have also exercised a pronounced influence on its city plan. At first, this was apparent mainly in the location of the few bridges which governed transportation. The possibilities in the mud-flats and marshes were not appreciated; in fact,

"The street system and the city as a whole turned their backs to the water, leaving it to the poorest, most casual development, or none; but recently all this has changed."

One of the first developments in this direction was the building of the beautiful Fens, then along the Charles River, and finally, including the Mystic and Neponset Rivers. Thus, five series of parkways formed "great lungs to the city's congested areas."

As to other important problems, according to Mr. Comey,

"From the beginning, Boston's docks and wharves gave a special character to the city as a port, but it is only in the past fifty years that any general scheme of port development has been pursued. To-day, most of the waterfront not devoted to recreation is developed or slated for development for wharfage for sea-going vessels."

Further growth is assured by the filling projects that have been carried out. One of the most recent and direct advantages of this development has been the "location of Boston's newest transportation agency—the principal airport—advantageously close to the heart of the city in East Boston."

* "Park System of the Boston Metropolitan District," by Edwin H. Rogers, M. Am. Soc. C. E., p. 2491.

GENERAL AERIAL VIEW OF BOSTON LOOKING WEST, SHOWING HARBOR, BUSINESS DISTRICT, CHARLES RIVER DEVELOPMENT, CHARLESTOWN AND CAMBRIDGE BEYOND.

AERIAL VIEW SHOWING STATE HOUSE AND BOSTON COMMON, CHARLES RIVER DEVELOPMENT BEYOND.

Boston's natural street layout did not lend itself happily to surface electric lines, so that,

"In 1898 the first American subway was built, since which time all surface cars have been removed from the center of the city, and rapid transit lines constructed to tap all the inner suburban areas."

*Recreation Features.*—Sanitary construction, however, as for sewerage and water supply, had almost no effect within the city proper. Outside, as in the Middlesex Fells Reservation, water supply blended naturally with the park development. Otherwise, stated Mr. Comey,

"Aqueducts have usually been buried and the land often left in private hands, although one or two of the newest constitute unused opportunities for through walking routes, extending many miles into the country.

"As distinguished from circulatory services, whether of people or of goods, land uses in Boston, really the end product of a city plan, may be divided into two main classes, public and private, and the former, in so far as they are important in the city plan, may be divided into recreational service and administrative service. Boston led the way in a connected series of parks, forming a real system and later two interconnected systems, an inner city system and an outer metropolitan system. Most of these were planned before the days of automobiles when the present service to traffic rendered by the parkways was unforeseen. The parks themselves have continued to increase in usefulness for recreation, although the outlying ones are still to-day in large degree what their generic name strictly means, 'reservations' for the future use of a population that may either need them more or appreciate them more."

Recreational features have also been supplied by the City, Mr. Comey stated. Beaches and inland ponds have been made available for bathing. Although often adjacent to school buildings, playgrounds in general are independent of the school system. School buildings in themselves have been made monumental features in the general architectural scheme for the city. This is also true of administrative buildings, of which the State Capitol is an outstanding example. The continuous domination of this feature has been made possible by insuring the present view for years to come by condemning the air rights in the district adjacent to the Capitol.

*Zoning.*—As early as 1904, Boston anticipated the zoning law movement by instituting a control over the height of buildings. The maximum at that time was 125 ft. in the business district, with a lower limit for the remainder of the city. The upper height limit was raised to 155 ft. in 1923, with the usual provisions for set-backs.

A year later the district was still further expanded into half a dozen or more divisions, with heights varying from 35 to 155 ft. The latest development is that by which a change has been made in "the maximum height limit to permit taller buildings within a pyramid-shaped prism starting 125 ft. above the ground, provided their volume did not exceed that previously permitted."

In all, thirty-nine communities are comprised within the City of Boston and its suburbs, and this condition has not lent itself to unified development. According to Mr. Comey:

"Each municipality has worked out its own salvation, often with no regard to others or to its own future as an integral part of a metropolis. Thus, radial

highways are numerous, although narrow, while cross-town or circumferential connecting highways are either lacking or imperfect, with numerous missing links. A number of years ago those services most imperatively demanding consolidation were so treated under metropolitan commissions."

Considerable unification has been obtained in both the sewerage and water supply development as described in other papers.

*State Authority.*—"Owing to the preponderating importance of Boston and the district in the State," explained Mr. Comey, "another factor in its development is still very much in evidence, namely, the State Government itself. * * *

"After years of piecemeal work a Division of Metropolitan Planning was set up, which now advises the Legislature on various large thoroughfare and transit projects for immediate relief both inside the City of Boston and elsewhere. It is also preparing a comprehensive metropolitan thoroughfare and transit plan, both as a guide for the Legislature and for guidance to individual communities. In 1929 another agency stepped into the field, the State Department of Public Works, with a plan for a radial highway northeast from the East Boston Tunnel, on which work is about to commence, through East Boston to existing highways in Revere.

"One result of this multiplicity of planning agencies, with overlapping and sometimes conflicting fields of activity, has been to stultify local planning work, except in Boston itself. Moreover, it has resulted in more than usual confusion and expenditure of human energy in getting anything done. Again, as no one body is yet charged with the duty of preparing a comprehensive regional plan, certain phases only are covered at all."

*Comprehensive Planning.*—In view of these many difficulties, Mr. Comey stated his belief that:

"Probably the most effective way to get ahead will be to charge the Division of Metropolitan Planning with the duty of preparing and keeping up to date a comprehensive metropolitan regional plan and of constantly presenting this plan before the Legislature and to local planning agencies. It would naturally confine itself to problems affecting more than one municipality and stimulate the local planning boards to develop and promote comprehensive city and town plans for their respective areas. Boston's progress in developing according to a plan has been great. Its opportunity for further similar progress to meet current urban needs is even greater."

## DISCUSSION

### TRAFFIC PROBLEMS IN RELATION TO THE PLAN OF BOSTON

By Robert Whitten,* Esq.

*Traffic Courts.*—In traveling from the North Shore to the South Shore it is natural to go through the center of Boston. There is no convenient by-pass to take traffic around the city. That was one of the planning problems described by Mr. Whitten. Another problem allied with it, was to enlarge the capacity of existing city streets. In order to plan new traffic arteries it was necessary to know where the traffic was coming from, where it would go, how

* City Planner, Cambridge, Mass.

much could be expected to use a particular route, and what relief the new route could be expected to give. Accordingly, stated Mr. Whitten:

"We undertook the problem of determining the amount of traffic moving daily between each of 69 traffic districts within the Metropolitan Area. More than 188 000 drivers were questioned as to where they came from and where they were going; and, as a result of the tabulation and analysis of that questionnaire, we were able to work out a formula which, with certain adjustments and modifications, could be applied to supplement the direct knowledge that we gained from our questionnaire. We determined the daily traffic between 2 373 pairs of districts, that is, the total combinations amounted to this number. We found that the traffic between these pairs of districts varied directly as the motor vehicle registration and inversely as the square of the distance between the districts. Then we made a careful estimate as to the probable increase in traffic, basing that estimate on the theory that traffic would increase in substantially the same ratio as the increase in the number of motor vehicles registered.

"We estimated that by 1960 there would be a doubling of traffic throughout the Metropolitan District. That is not a very great increase, because we have seen traffic increase five or ten times in the last fifteen or twenty years; but a doubling of the already existing congestion is quite a serious problem and really involves a different sort of highway planning from that to which we have been accustomed in the past."

*Express Highways.*—To keep abreast of traffic requirements by simply widening one street after another was too impractical and costly, in the opinion of Mr. Whitten. The construction of express roads was recommended as the best way to cope with the increasing seriousness of the traffic problem.

The result of the traffic survey was given by Mr. Whitten as follows:

"The motor vehicles in the thirty-nine cities and towns in the Metropolitan District, including Boston, produced a mileage per day in 1927 of 4 952 000 vehicle-miles, or a yearly total of 1 800 000 000. The unit price of 10 cents per mile is a fair estimate of the cost of motor vehicle transportation when all kinds of cars are included as well as those who use a car a great deal and those who get only a small mileage out of it. At 10 cents per mile, the total is $492 000 per day or $180 000 000 per year; so that that is the amount that the people of the Metropolitan District are spending on motor vehicle transportation. It is clear that any expenditure that will materially improve transportation facilities for the users of motor vehicles, can be economically justified."

There are other factors to be considered, however, before an express highway can be economically justified. One of the most important of these, according to Mr. Whitten, was the problem of preventing injury to residential property by making extra expenditures in the interest of beauty and attractiveness.

## ELEVATED PEDESTRIAN WAYS

By A. L. Kasson,* Esq.

A thought in connection with elevated streets, was given expression in a written discussion presented to the Division by Mr. Kasson. His idea was to separate pedestrians from automobiles by constructing elevated sidewalks. As an outline of the fundamental requisites for the plan of the future great city, Mr. Kasson presented the following items:

* Cons. Engr., Mattapan, Mass.

1.—An inner electric zone—all electric transportation.
(*a*) The minimum of exhaust gas with its dangerous carbon monoxide content.
2.—Co-ordinated transportation facilities.
3.—Underground high-speed radial and belt electric railways, with automobile parking space at outer stations.
4.—Underground pipe and wire tunnels.
5.—Surface gas and electric buses.
6.—Elevated pedestrian deck—clean and safe.
7.—Three entrances to stores from subways, surface streets, and elevated pedestrian-decks.

## THE INFLUENCE OF CAPITAL CITIES ON OTHER CITIES IN THE STATE

By John Nolen,* M. Am. Soc. C. E.

This seems to be an engineering world, Mr. Nolen acknowledged:

"But," he continued, "I believe it can only remain an engineers' world provided we manage to deal more directly than we do now with certain of the ends and aims of life rather than with the means of life; and in our cities the question of public buildings, grouping of public buildings, and all that has to do with the dignity, the glory, and the beauty and satisfaction of life is involved in such questions, which might be referred to as the emotional end and which could readily be illustrated.

*"Centralization."*—There is another problem of how far we can bring together public buildings and administrative groups, without getting into new dangers and difficulties involved in centralization. The problem of decentralization, I notice, is quite different than it was a few years ago. The Boston that is being discussed is not really the City of Boston, but the Metropolitan Area. There is not much possibility in discussing Boston in its strict political sense in planning matters as a city.

*"Comparison with European Capitals."*—As to the question of the inspiration and guidance which a capital city may have, it is very constructive to look at Washington in studying the problems of our State capitals. The Hon. James Bryce, in his 'American Commonwealth' says that the United States is the only great country in the world which has no capital, and he says, 'By a capital I mean a city which is not only the seat of political government but also by its size, wealth, and character of its population, the head and center of the country, the leading city of commerce and industry, a reservoir of financial resources, a favored residence of the great and powerful, a favored spot in which the chiefs of the learned professions are to be found, where the most potent and widely read journals are published, and where men of literary and scientific capacity are drawn'."

Boston comes closer to fulfilling this definition of a capital than almost any other American city. Mr. Nolen stated that it compared favorably, in this respect, as least with such secondary European capitals as Dresden, Budapest, Versailles, Brussels, Prague, Stockholm, Zurich, or Geneva. If it had not been a capital city, under the influence of State initiative in providing large metropolitan improvement, the history of Boston as regards city planning would have been very different.

City planning, with a State capital as its laboratory and the resources of the State as its support, could develop constructive theories on public safety,

* City Planner, Cambridge, Mass.

health, convenience, nature, art, prosperity, and happiness. The capital city would then serve as an example to other cities. In some respects, Washington has done practically all these things.

*Desire for a Better City.*—Continuing his interesting discussion Mr. Nolen remarked:

"Boston is a capital city and, as a metropolis and a pioneer in municipal regional development, it has made a creditable showing. Like all American cities, it has great needs and larger possibilities; perhaps its greatest need is more complete and effective machinery, both official and non-official. That is the striking thing in Chicago. You see them working hand in hand, official and unofficial citizens' movements to get things done, the working out of comprehensive regional plans and their orderly execution.

"Not only can one American city profit from seeing the progress of another, but all can gain by a study of the planning projects in Europe. As a matter of fact, the most ambitious proposals for American cities in many ways do not equal the actual achievements of old-world cities. In Europe, especially on the Continent, there is better provision for city life, for business, for health, and for pleasure. Consider, for example, the lake cities of Switzerland, such as Zurich, Lucerne, and Geneva. Go about in their public places and without exception we shall find orderly development of water frontages for public use, a rational street system, and relative freedom from nuisance, regulation of wires and railroads, and careful public planning, as well as an abundance of recreational areas, public gardens of all sorts, practical sites for public buildings, art galleries, museums, comfortable and sanitary housing in many instances, and all that pertains to a consideration of future needs.

"One of the members of the faculty of Harvard said years ago that the people of Boston can have as fine a city as they want, providing they want it badly enough to be willing to pay for it. Nothing as good as a fine city is to be had for nothing. If the people of Boston decide they have not time to make their city what they would like to have it be, it will mean that they have other things they would rather do with their time than improving their city. If they decide that they have not the time or energy to make their city what it ought to be, it will mean there are other things they prefer and to which they would rather give their energy and money."

*State Pride.*—The people should have the same sense of pride and loyalty toward the capital of their State as they have for the capital of the Nation. Unfortunately, this is not the case except in a few cases, according to Mr. Nolen:

"Instead of that there is a sense of envy and pettiness, generally speaking, in the people of the State toward their State capital. There is not only no affection, but there is no good sense and no loyalty. There are a few exceptions. In the State of Wisconsin the City of Madison is an exception, and there are others, especially where the State universities are located, where they are centers of culture and education as well as centers of government."

## CITY PLANNING EDUCATION FOR THE PROFESSIONAL CITY PLANNER

By H. V. Hubbard,* Esq.

*A Profession by Synthesis.*—Out of his experience, Professor Hubbard described present practice in designing college courses in city planning. This, he declared, was not offered as a final statement of what the courses should be, but of what they are at present. To quote him directly:

* Prof. of Landscape Architecture, Harvard Univ., Cambridge, Mass.

"The teaching of any profession comes after the practice of it. There must be enough public demand for knowledge and skill in any field, so that a man may earn a reasonable living in it, before many people will spend time and money in preparing themselves professionally in that field.

"Up to recent times, the development of new professions and the attendant teaching has been similar in each case. The new profession has been something split out from a previous broader profession. In the old days one man might be architect, engineer, sculptor, painter. Later it was all a man could do to master engineering. Then, as this field widened and detailed knowledge increased, we had civil engineers, mechanical engineers, electrical engineers, and so on.

"But city planning is different in this regard. It has come about not by division but by collection. It is the result of synthesis rather than of analysis. The city planner must be intelligent in the fields of engineering, architecture, landscape architecture, law, government, and many others. Plainly, he cannot know more than one of these subjects as a practitioner can know it who spends his life on that alone. What then can he honestly claim to know without pretending to a superhuman wisdom?"

*What City Planning Is.*—As an answer to his own question, Professor Hubbard offered the definition that, "city planning is the official record, of the will of the community as to how it means to use the physical areas which it controls." In other words:

"When a community wakes up to the fact that it is one single big business, that it cannot plan intelligently about its streets alone or its buildings alone, or its sewers alone, or its parks alone, but must consider all these and many other things before it can reasonably decide about any one of them, then it knows that it wants city planning.

"What then is the city planner? He is not an inspired prophet, to whom all wisdom as to cities has been revealed. Rather he is the trained recorder of the decision of a big committee—the man who can understand how all the requirements of the community may fit together to make one workable scheme, and who has the technique to record the decision, in words or on plans, so that the scheme may be worked out, consistently but flexibly, by those who come after him.

*"The Evolution of City Planning.*—Here it should be noted that one of the essential requirements is the appearance of the result. The planner must be an artist. His profession is a fine art.

"Now it has come about very naturally, with the growth of the demand for this sort of service, that almost all the so-called city planners have been first something else. They were, and are, mostly engineers, architects, landscape architects, and so on, trained to co-operate, trained to think broadly in the larger field of city planning. The reason that so many city planners have come from the small profession of landscape architecture is, I believe, that the demand found them among the few who were already skilled in designing outdoor areas for use, and at the same time for beauty, and in their work with park systems and land subdivisions they had already been thinking in city planning terms. But this does not mean that, therefore, city planning is a part of landscape architecture, nor is it a part of engineering or of architecture. These professions have a heavy share in city planning because they are the plan-making professions, but city planning is broader than they, and is the result of their co-operation with one another and with many other professions and businesses."

*City Planning Courses.*—For some time past various schools of architecture, engineering, and landscape architecture have given instruction in city planning, with the purpose of opening the eyes of students to this large op-

portunity for service in connection with their own particular specialty. For example, the courses at Harvard University began in this way in 1909 until now it has a separate School of City Planning. The idea is to give the degree of Master in City Planning at the end of a 3½-year course following a Bachelor's degree.

The purpose of such a comprehensive course should be threefold: First, to train a few men to be professional city planners; second, to train many men of different professions to be efficient co-operators in city planning; and, third, to encourage research along city planning lines on subjects that the ordinary practitioner has not the necessary time to study.

Three principles were recommended by Professor Hubbard as guides to conducting research in city planning. In his own words:

"In conducting research, the school, after seeking advice from those who have the most intimate acquaintance with each subject considered, should assure itself (1) that further knowledge (or accessibility of knowledge) of this subject would be of real use in city planning; (2) that the benefit of the knowledge obtained would be commensurate with the expense of the research; and (3) that sufficient publicity and accessibility be given the results so that these may be used while still capable of application."

Engineers as a class are often rebuked for not being able to express themselves in good English. This is an especially important subject for students in city planning, in the opinion of Professor Hubbard.

*Content of Studies.*—An outline for a four-year course was given, as follows:

For the first year—mechanical drafting; freehand drawing; lecture courses on the theory of design in city planning; history of mediaeval, renaissance and modern art, and principles of landscape architecture; principles of architecture; topographical surveying; drafting-room course on principles of construction, etc.; horticulture; and a lecture course on the historical development of city planning.

For the second year—freehand drawing; advanced construction drawing; municipal construction; municipal planning design; social aspects of town planning; advanced lecture course by active practitioners; and a drafting-room course in city planning design.

For the third year—modeling; materials and methods of construction, with weekly inspection trips; lectures on professional practice and contracts and specifications; municipal government; and municipal administration.

For the fourth year—advanced city planning design and thesis.

## CITY PLANNING EDUCATION FOR PUBLIC OFFICIALS AND THE GENERAL PUBLIC

By Charles R. Gow,* M. Am. Soc. C. E.

*Attitude of Public Officers.*—There are different concepts as to the real meaning of city planning: That of the city planner; that of the political appointee; and that of the general public. Said Colonel Gow:

* Pres., The Gow Co., Inc., Boston, Mass.

"In large measure the elected representative of the people is inclined to look upon city planning and the city planner with feelings of mingled fear and suspicion; fear in the first place that the city planner, in his zeal and enthusiasm for certain ideas, will disregard entirely the economic aspects of the case and that he will undertake to impose upon the public a greater financial burden than it is justified in assuming. In spite of the protestations of the expert technician and the need of certain improvements, the elected representative feels that there lurks somewhere in the background some sinister or unworthy motive which is being put forward and clothed in the robes of virtue by this plausible sounding expert who presents the case before the public body.

"That is entirely natural; it should be expected and should be recognized by every man who undertakes to develop and introduce municipal and public improvements in comprehensive form, because the elected representative of the people is constantly subjected to continuous pressure from various sources for his support for various schemes and improvements. If he were to give ear to even a small percentage of the total he would very shortly exhaust the financial possibilities of the political entity which he represents. Therefore, he must necessarily scrutinize with great care and even with suspicion the proposals which are put before him."

*Paying for Future Benefits.*—This viewpoint should be recognized in attempting to educate public officials. Furthermore, the city planner must be prepared to answer some perfectly natural questions that arise, one of which is:

"Why should I of this generation concern myself with what is going to happen fifty or one hundred years hence or to expend, out of my savings, money for the ultimate benefit of those who will live in subsequent generations?"

The possible answer to this question must be kept in mind. To quote Colonel Gow:

"When you are selling a comprehensive plan to the public you are going to be met with that attitude of resistance against adopting some bold and comprehensive forward-looking scheme, the major benefits of which are going to inure to some future generation of citizens; and that is a real obstacle. It is very difficult to sell that idea to the taxpayer who is already seriously overwhelmed with requirements of a public nature and who is naturally resistant against any such thought.

"In these forward-looking movements we ought to talk more in generalities about the future of the scheme and more in detail about its immediate phases and its particular benefits as they apply to the present period. I think to approach the problem in that way will more likely produce results than to elaborate on the glories of the time to come when some of our successors will enjoy great benefits because we have contributed from our current income for this very desirable improvement.

"Furthermore, I think there is a general appreciation on the part of all citizens that even though we might, in our present wisdom and judgment, provide a very comprehensive plan for the benefit of some future generation, we cannot commit them to that plan, and that when the time arrives for them to carry on the development of the plan they will very likely prefer to exercise their own judgment, whether it be superior or whether they merely think it is superior to ours. In any event, that is a difficulty that must be considered in approaching these major problems of long-time development."

*Public Inertia.*—There are other problems worthy of consideration. For example:

"When you come to suggest improvements for rebuilding an existing community, you get into a great many very practical difficulties. In the first place, people become accustomed to their surroundings, which gather a certain sentimental value; they attach significance to the fact that some old disreputable structure that they have looked at all their lives has become an old familiar friend, and they dislike to see it go.

"They have accustomed themselves to certain movements and methods that have become second nature, and it means more or less of a disruption in their ordinary means of life and locomotion if those are changed. There you have what might be called a public inertia to overcome, to present a sufficiently attractive picture as far as the improvement is concerned, to stimulate their enthusiasms for it and subordinate that perfectly natural inertia which most of us feel about old associations.

* * * * * * *

*"Sovereign Rights.*—Probably the greatest stumbling block that the expert city planner encounters in his efforts to introduce modern ideas into various municipalities and communities is that which is introduced by the volunteer expert. That is a peculiar thing. If a private enterprise were to hire you or me to advise it respecting a technical matter with which it felt it was not qualified to deal, the chances are very good that it would follow in the main our advice so far as it felt capable of doing it and so far as it did not exceed the financial resources allowable. At least, it would accept our judgment as final on that particular problem."

That, however, is not so when one is dealing with the public, in the opinion of Colonel Gow. Each individual then reserves the right to become an expert in his own name and, therefore, when a well developed program is presented before some public tribunal, the sovereign right of the great American citizen comes into play. Quoting Colonel Gow:

"He reserves the right to rise upon the floor and to pick your suggestion to pieces and to point out in his ignorance where it is all wrong. And the public mind is so constituted that it is very easily confused. It has great difficulty in discriminating between the qualified expert whose experience and training and judgment have warranted placing considerable reliance on his views, as distinguished from the irresponsible, self-appointed expert who merely injects himself into the picture because he feels, even without any training or experience, that he can see this thing so much more clearly than you, that your proposition looks ridiculous to him.

"I have noticed on innumerable occasions how very meritorious proposals are sacrificed before our legislative bodies because of that fact and because of the confusion which results in the minds of the responsible representatives who have to pass on it and the confusion of mind of the general public who are unable to discriminate between the several responsibilities of the individuals who have expressed themselves."

*Help by Individuals.*—To counteract this influence, Colonel Gow suggested that all who are interested in a city planning project could do much to help. For example, if a qualified expert has been chosen to deal with a certain public problem, persons interested in the success of that problem cannot afford to attack the general decision unless it is so obviously wrong as to warrant the most drastic measures. The only remedy is for city planners to come to each other's assistance when they are attacked by irresponsible and inexperienced pseudo-planners.

Speaking of local changes, Colonel Gow stated that:

"There is another factor in human character that has to be dealt with, and that is the fact that no general public improvement can usually be brought to fruition without sacrificing in some measure the property rights of individuals; and there is a large element of sympathy in the community toward men who would be deprived of any rightful use of their property or appreciation in its value. That is not because the people are interested in that individual's welfare so much as it is that they see that the extension of that principle will mean eventually that they are going to be among the victims themselves."

One phase of the subject upon which city planners might well concentrate considerable effort is highway development, which may be classed as an absolute essential.

*The Future.*—In the summary of his paper, Colonel Gow stated:

"A movement such as this, which has been crystallizing rapidly in the last few years, for the recognition of city planning as an independent art of engineering, is to be credited with a great deal of progress. It is not to be expected that in the brief period which has elapsed since this movement started, very marked progress should have been made. The fact is that we are more and more coming to realize it as a legitimate and necessary activity, and as time goes on * * * it is going to be possible for city planners in general to concentrate more and more upon the needs of our several communities for improvement and gradually to wear down a resistance to city planning improvements.

"I look forward to the time when city planners can go before the various public bodies with full backing of their professional associates and create the impression of knowledge and general understanding of the problem from both angles, from the technical and from the public standpoint, which is demanded if any progress is to be made."

## DISCUSSION

### INTERESTING PUBLIC OFFICIALS IN CITY PLANNING

By Karl B. Lohmann,* Esq.

*Methods of Progress.*—The importance of the subject assigned to Colonel Gow was stressed by Professor Lohmann in a written discussion read by Harold M. Lewis, M. Am. Soc. C. E. Quoting verbatim:

"When properly enlightened, the citizens are much more easily persuaded to favor a high standard of improvements. Arousing the interest of the community and educating the public are accomplished in many ways. Important is the experience of cities, such as Chicago, where large programs have been sponsored by the general public as a result of being educated to the planning needs of the city. No less important is the work in the elementary, secondary, and higher centers of learning where the educational work in city planning has taken the form of prizes for essays that treat of city planning subjects; of civic books for the schools, as in Dallas, dealing especially with the city planning proposals and accomplishments for the City of Dallas; or of appreciation courses in city planning in our colleges and universities. Like-

* Prof. of Landscape Architecture, Univ. of Illinois, Urbana, Ill.

wise important have been the activities of civic committees in various parts of our country in supplementing the work of city planning commissions by educational methods.

"The opportunity of membership on plan committees indirectly furnishes a valuable form of education for many persons. Excursions of public officials to other cities to inspect city planning accomplishments of interest have been arranged in some instances with notable value to the persons who have made such pilgrimages. Newspapers, magazines, moving pictures, and radios, indeed, all have played a part, and still may be used in a city planning direction to further the education of both public officials and the public."

Professor Lohmann warned against creating the impression that city planning is a "hashed" profession, or one that is "docked, lopped, or truncated." It should be considered an aggregate profession, with many aspects, all of which should be "integrated into a single important structure."

*Undergraduate Courses.*—Turning then to the subject of Professor Hubbard's paper, he stated the relative value of post-graduate and under-graduate teaching of city planning. To quote briefly:

"The advantage of graduate school work in city planning is that it compels foundational preparation which is highly valuable for any one who is electing a specialty of this kind. Moreover, it lends itself more conveniently to the pursuit of research activities which are so important to be considered as part of the training in city planning. However, we need not infer from this that full undergraduate courses in the subject of city planning should necessarily be barred. Undergraduate courses at least bring the training within the reach of those who for financial reasons might be obliged to forego it.

"Where options in city planning are offered under such courses as civil engineering or landscape architecture and architecture, caution should be exercised that the curricula do not become one-sided. Although a foundational preparation in one of the several subjects contributing to city planning is much to be desired for any one specializing in city planning, yet every care must be taken not to neglect fair and proper consideration of the other contributing phases.

"Perhaps we shall continue for some time to have engineering city planners, architectural city planners, landscape architectural city planners, and so forth. As time goes on, however, it is reasonable to suppose that a better definition of the limits of city planning, or at least a more general agreement and understanding as to its sphere, will lead to placing it on legs of its own as a separate independent profession while it preserves its understanding relationship to its brother and sister lines of work."

# STRUCTURAL DIVISION

OCTOBER 10, 1929—9:40 A. M. TO 12:30 P. M.

Attracted by an interesting program, a large and enthusiastic audience was awaiting the opening of this meeting. The first paper covered quite completely the design and construction of the recently completed Lake Champlain Bridge. The author, Professor Spofford, has kindly furnished a digest which is here printed in its entirety and gives a bird's-eye view of the project.

## THE LAKE CHAMPLAIN BRIDGE

By Charles M. Spofford,* M. Am. Soc. C. E.

The growth of motor traffic in the United States has resulted in the construction of numerous toll bridges, generally built by private capital, but occasionally by public bodies. The recently constructed bridge across Lake Champlain between the States of New York and Vermont furnishes an example of the latter type.

*Economics.*—This bridge was financed, built, and owned by a public commission set up by the two States of New York and Vermont, under authority of Congress, and having full power over the particular project entrusted to it. The total cost of the bridge, including not only that of its construction and the real estate for its approaches, but also the administration, legal, and engineering expenses, was somewhat less than $1 200 000, of which $1 000 000 was raised by the sale of 30-year bonds bearing 4½% interest. The income from tolls is to be used in liquidating the cost of the bridge so that it may be made free as soon as feasible.

All estimates show that the income from tolls should be sufficient to meet the requirements of the bond issue long before the bonds mature. These estimates are confirmed in some degree by the fact that in the five weeks after the opening of the bridge, ending September 30, 1929, the receipts from tolls were $20 000. Had it been necessary for the two States to appropriate funds to pay for the bridge outright, without any prospect of obtaining revenue therefrom, its construction would probably have been long deferred.

*Location and Type.*—The new bridge is the only one crossing Lake Champlain in a length of 120 miles, a portion of the lake which has always constituted a serious barrier to intercourse between the States of New York and Vermont. In times of open water, transportation for vehicles across the lake has been available, to a limited extent, by ferries, but most of these were operated only by day, some of them had hazardous approaches, and, in some

* Hayward Prof. of Civ. Eng., Mass. Inst. Tech.; Cons. Engr. (Fay, Spofford & Thorndike), Boston, Mass.

cases, they were not operated during bad weather. In winter, occasional roads have existed across the ice.

The Lake Champlain Bridge is located at the site of one of the ferries which has discontinued service. It will doubtless reduce traffic on other near-by ferries; on the other hand, it is expected to develop new traffic, as has been the case with other improved traffic routes across waterways, such as the Philadelphia-Camden Bridge across the Delaware, the Bear Mountain Bridge across the Hudson, and the Holland Tunnel. Such additional traffic will probably consist, to a considerable extent, of through traffic between Northern New England and Northern New York, for which the bridge will furnish a short and convenient route.

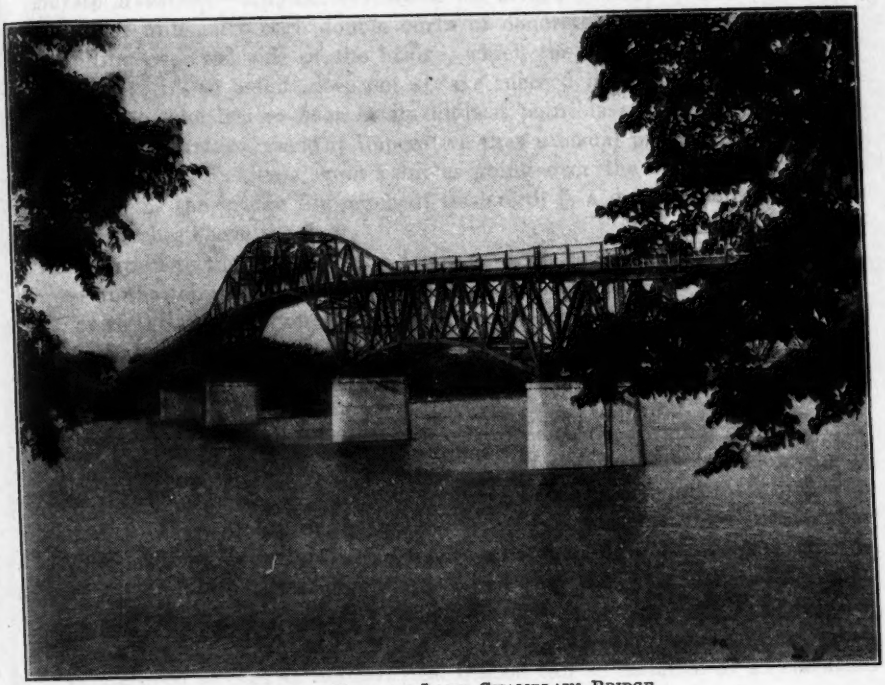
From an engineering standpoint, the bridge has several features somewhat out of the ordinary and worthy of note. The great steel trusses are of the continuous type which has not been much used in the United States before, so laid out that the center span has the appearance of an arch, but without the serious disadvantages which would result from the large horizontal thrusts an arch would produce. These thrusts would be of such magnitude that, to carry them to bed-rock at the great depth at which it lies at the bridge site, would require large and expensive piers, the cost of which would be excessive.

*Piers.*—The method of building the bridge piers in the deep lake water involved the construction of large steel boxes open at the top and bottom—"open coffer-dam" is the technical term for such construction. The sides of these boxes were formed of interlocking steel piles driven to bed-rock. After the piles were driven, the mud within the coffer-dam was dredged out, that adhering to the steel work being loosened before the dredging was completed by powerful hydraulic jets. The removal of the mud and its replacement by the lake water filtering through the walls of the coffer-dam resulted in the samples of water from the interior of the coffer-dam being freer from sediment than that of the lake water outside.

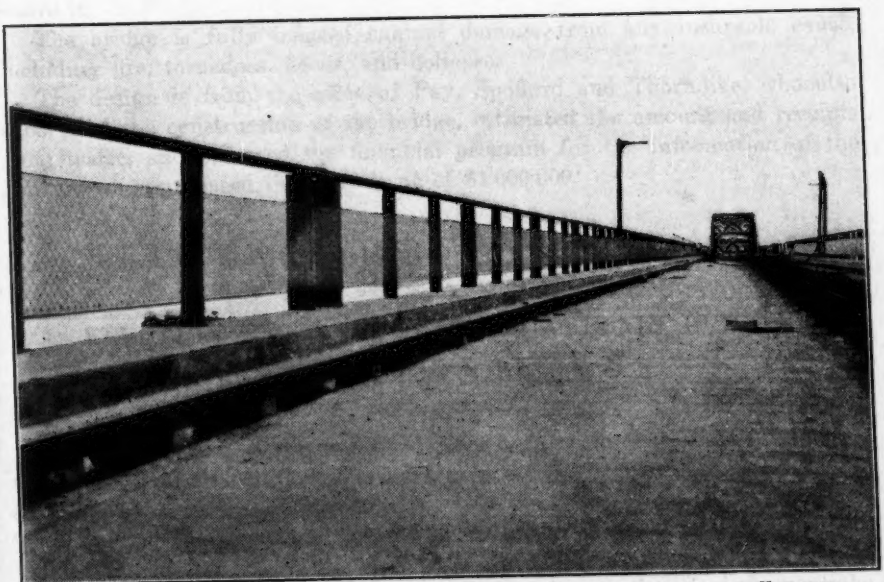
After the removal of the mud, the rock bottom enclosed within each coffer-dam was examined by divers to make sure that it would be suitable for supporting the pier; the concrete for the pier footing was then deposited by buckets which automatically opened upon striking either the rock bottom or the concrete previously deposited. Above the mud line the water was pumped from the coffer-dam and the remainder of the concrete was laid in the open air.

*Braced Coffer-Dam.*—To resist the pressure outside the coffer-dam, which, on account of the greater weight of the mud, would not be entirely balanced by the water within, the coffer-dam was braced by strong steel frames placed in it in the form of a nest as soon as some of the mud was excavated. These frames were then lowered and put in place one by one as the remainder of the mud was excavated. They were ultimately buried in the concrete, the coffer-dam being made so large that the concrete containing the frames was outside the area of that required to support the bridge structure.

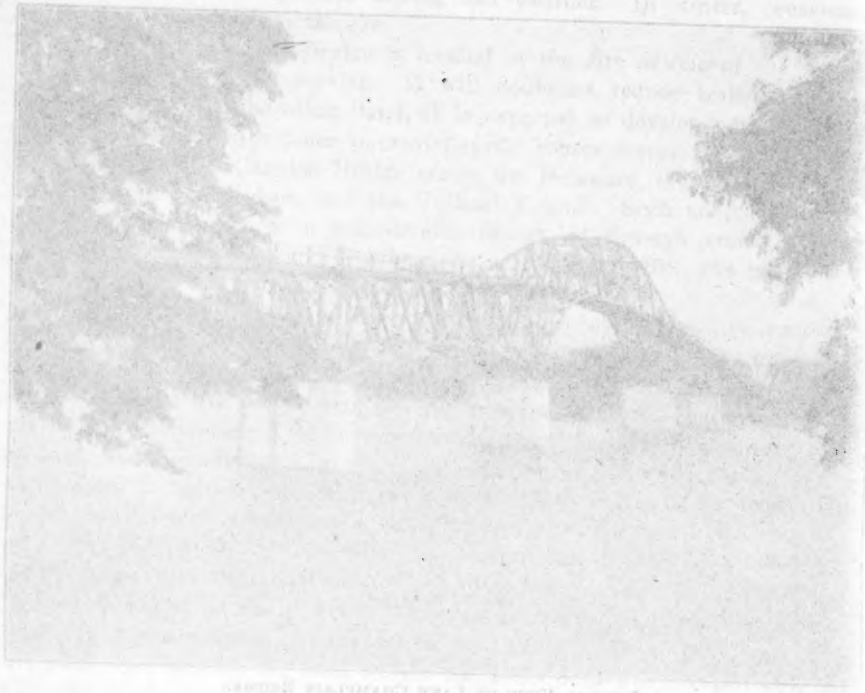
So far as is known, the construction of bridge piers by single wall coffer-dams of this type had never before been used to a depth so great as that at the Lake Champlain Bridge, where the bed-rock is in places 100 ft. below the surface of the water.



GENERAL VIEW OF LAKE CHAMPLAIN BRIDGE.



LAKE CHAMPLAIN BRIDGE; VIEW SHOWING FLOOR. NOTE HIGH CURBS AND FENCING.



*Safety Measures.*—Another feature of the bridge worthy of attention is the provision of unusually high double curbs of concrete to prevent automobiles from falling over the side of the bridge, which the ordinary curb from 6 to even 10 or 12 in. in height does not always succeed in doing. The fact that the surface of the bridge floor at its highest point is approximately 100 ft. above the lake surface, made it imperative that unusual precaution should be taken to prevent accidents from vehicles going over the side of the bridge, particularly as the bridge for much of its length is a deck structure without trusses or arches above the floor.

In addition to the high curbs, a fencing which consists of steel wire mesh, secured at the top and bottom by strong wire cables connected to steel railing posts, or to the trusses where they rise above the floor in the center span, is of itself probably sufficiently strong and elastic to hold back any car striking it. Those responsible for the bridge believe, however, that no cars will ever ride the curb.

*Other Details.*—The roadway surface is of concrete, forming a part of the main floor-slab, but laid in such a way that if damaged by traffic it can be replaced without injuring the main concrete slab, the strength of which is a vital part of the strength of the bridge. This is accomplished by introducing a layer of wide-mesh burlap in the concrete during the process of laying, thus serving to establish a line of cleavage such that the concrete above can be readily removed without injury to the main slab itself.

The bridge is lighted by electric street lights, has electric navigation lights, and is equipped so that an aeronautical beacon can be installed at an elevation of approximately 135 ft. above the lake level if air navigation interests desire it.

The bridge is fully insured against damage from any insurable cause, including fire, tornadoes, floods, and collapse.

The design is from the office of Fay, Spofford and Thorndike, who also supervised the construction of the bridge, estimated the amount and receipts from traffic, and prepared the financial program for the information of the bankers who purchased the bond issue of $1 000 000.

## DISCUSSION

### ERECTION PROBLEMS—LAKE CHAMPLAIN BRIDGE

By H. W. Troelsch,* M. Am. Soc. C. E.

On one end, two of the approach spans were made continuous. Of these, the one nearest the shore was erected in a normal manner, as shown in the diagram, using falsework throughout.

*Falsework Tower.*—Then, for the second span, as Mr. Troelsch explained, "We cantilevered from the first pier out to the sixth panel, and at that point erected a large falsework tower. This span was then jacked up at the

* Asst. Engr., Am. Bridge Co., New York, N. Y.

tower until the stress in the top chord over the pier was relieved. The chord was disconnected to avoid anything indeterminate, and erection was continued by cantilever construction to the further pier. There the end was raised and as soon as the falsework tower was free, it was cut down. The end was then lowered until this chord member was in place, and then was raised until the jacks showed the calculated reaction for the load then on the span."

The main central span with its adjoining side spans was also continuous. This part "was erected by placing falsework under the two side spans, and then by cantilevering out to the middle."

In driving the temporary pier, a steam hammer was suspended from the boom of a traveler above on the outer end of the partly completed span. Then,

"Instead of using leads, the piles were guided by means of cables stretched from the completed portion of the span over to the next pier, with wooden guides lashed across it. There was ice on the lake at the time and that served as a working platform for the men, although it was not of much use for guiding the piles."

*Spliced Piles.*—The piles themselves were made and driven in sections. When the driving head came to a convenient position, the next section above was added and the driving continued until another section could be applied.

As to the spliced end, Mr. Troelsch referred to the sketch and explained that,

"The piles were sawed across on the square, the four angles were bolted to each corner with ¾-in. bolts running in both directions. The use of squared timber we find gives us a straight member which is more dependable in column action, and is not as subject to bending as a pile with a certain amount of initial curvature in it, has a larger area, and has a greater salvage value. We usually save these square piles, and can sell them for quite a considerable sum."

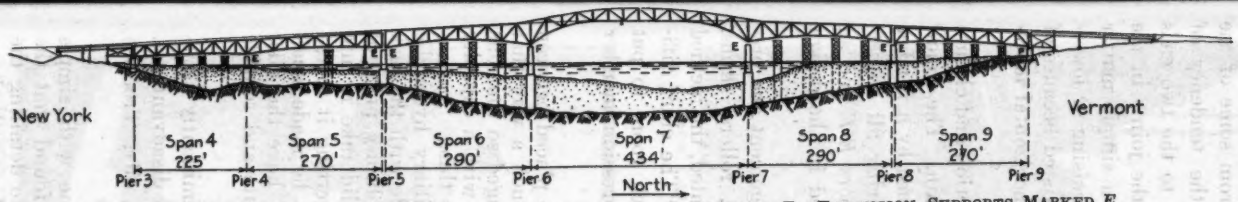
*Closure Operations.*—Making the closure on the main span gave an opportunity for a nice adjustment. Mr. Troelsch pointed out the general relation of parts as shown in the diagram:

"The ends of the span at *A* were set down about 20 in. The effect of that was to rotate the entire half of the structure about Point *B*. Since Points *C* and *D* were above this they were moved back, which caused an opening at the top and at the bottom."

The deflection had to be taken into consideration, also the temperature and the load on the respective trusses. After one series of computations and adjustments, as Mr. Troelsch related,

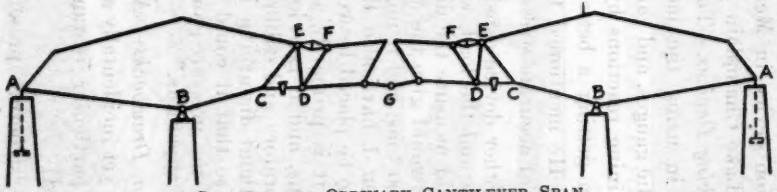
"We had an opening of about 4 in. at the bottom and about 5 in. at the top. We moved the whole south half of the structure on the Vermont side to the north 4 in., which closed the bottom. That should have left an opening of an inch at the top. Strange to say, when we measured it we found we had about 2¼ in. The only way I could account for it was that the members sliding between the gusset-plates had met with considerable friction or resistance, and acting on the lever arm to the depth of the whole span, had caused a bending back. These spans are rather flexible, and it might very well be that such a bending was produced.

"Just at that stage of the operation, however, before we were about to close the top, a heavy wind storm arose, and we had to quit work for the day. The next morning we measured the opening and instead of 2¼ in., which it

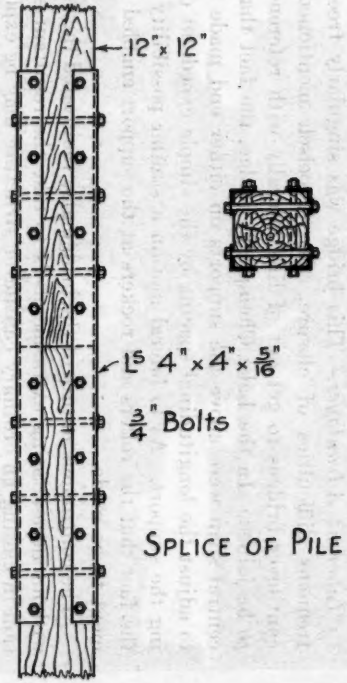



BRIDGE ELEVATION, SHOWING FALSEWORK. FIXED SUPPORTS MARKED *F*; EXTENSION SUPPORTS MARKED *E*.

CLOSURE FOR LAKE CHAMPLAIN CONTINUOUS BRIDGE.



CLOSURE FOR ORDINARY CANTILEVER SPAN.




DETAILS OF CANTILEVER BRIDGE ERECTION.

was the night before, it had settled down to 1 in., which was exactly what we expected it to do. The rest of the operation was simple, consisting of raising Points *A* and *A'* to bring the middle ends together."

*Inherent Advantages.*—This bridge was singularly free from some of the frequent difficulties of closure. Mr. Troelsch mentioned the tendency of cantilever bridges to get out of line laterally with reference to the two ends to be joined. In the Lake Champlain Bridge, the fact that the joints in the central span were above the supports on either end made it a simple matter to adjust the longitudinal spacing by the simple expedient of raising or lowering the support. As to lateral motion a similar possibility existed because of the fact that the rollers and rockers at the support enabled a horizontal movement to be made.

The continuous nature of the bridge also facilitated the erection and closure without necessitating extra members to take the resultant strain. The condition in closing an ordinary cantilever structure may be explained by a further sketch, showing that the shoes at Points *B* are fixed, requiring all adjustments in the suspended span. Toggles are inserted in Members *E-F* and an expansion device in Members *D-C*. "All of that was avoided in the design of the Lake Champlain Bridge."

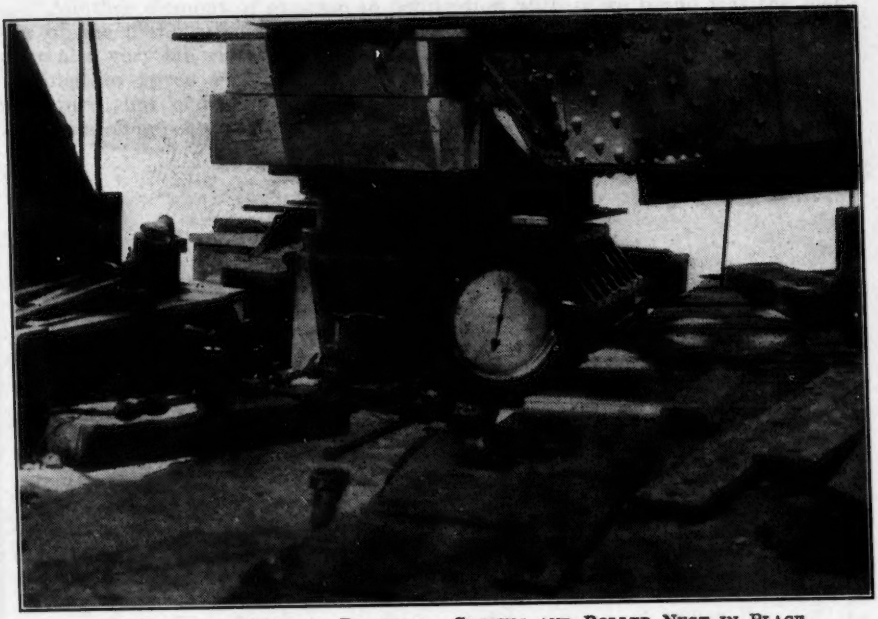
*Jacking Devices.*—Then, Mr. Troelsch illustrated the arrangement and procedure in using the jacks, the necessity for castors, shoes, rollers, shims, hydraulic gauges, and various other devices. Admittedly, he stated, the method of obtaining reactions by jacking, left much to be desired. "The chief difficulty in getting a better appliance is that there is not much room to put it in." He mentioned the possibility of using a short compression member capable of accurate stress measurements.

"Another device consists of an elastic ring which could be placed between the jack and the load, with a micrometer attached to it in such a way that you could measure the distortion of the ring. It can be arranged so that one reading would give the answer. Such a thing has been made with a capacity up to 100 tons, and it might be available for such situations as this.

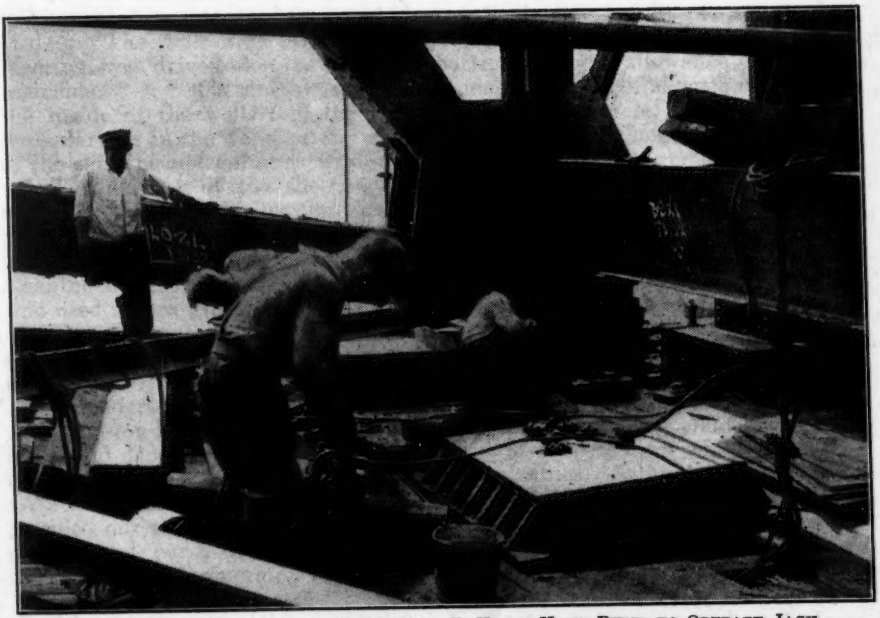
"Then I have also thought of the feasibility of an auxiliary hydraulic device to be placed between the jack and the load. The trouble with the jack is that it is primarily a lifting device, a rather heavy and clumsy piece of apparatus, and it gets knocked around in the field. If you did take it into the laboratory and calibrate it, you would have a hard time getting it to the field without disturbing it a lot. This auxiliary device might be made small enough so that it could be readily handled. You would still have the difficulty of the pressure-gauge, which is not an especially accurate and reliable piece of apparatus."

*Minor Drawbacks.*—All in all this type of structure is eminently satisfactory; yet incidentally a continuous structure has a few minor disadvantages. In the particular instance of the Lake Champlain Bridge, Mr. Troelsch explained:

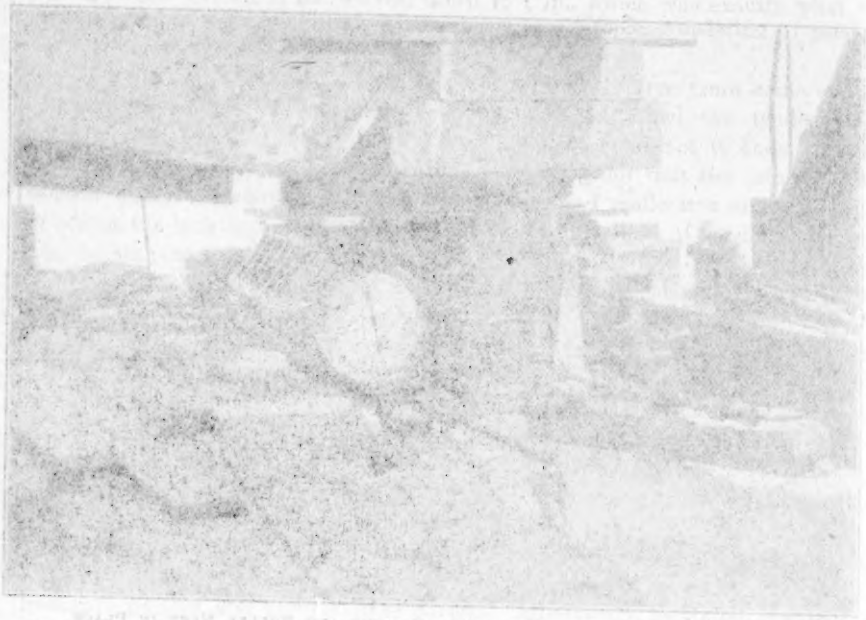
"The structure presents a more difficult problem than one with simple spans, inasmuch as the two ends are not symmetrical. We found that the drawing-room force was thoroughly schooled and accustomed to detailing the ordinary simple span, and that a structure of this sort required a much closer supervision. The men ran into unfamiliar conditions, and everything had to be watched much more closely.

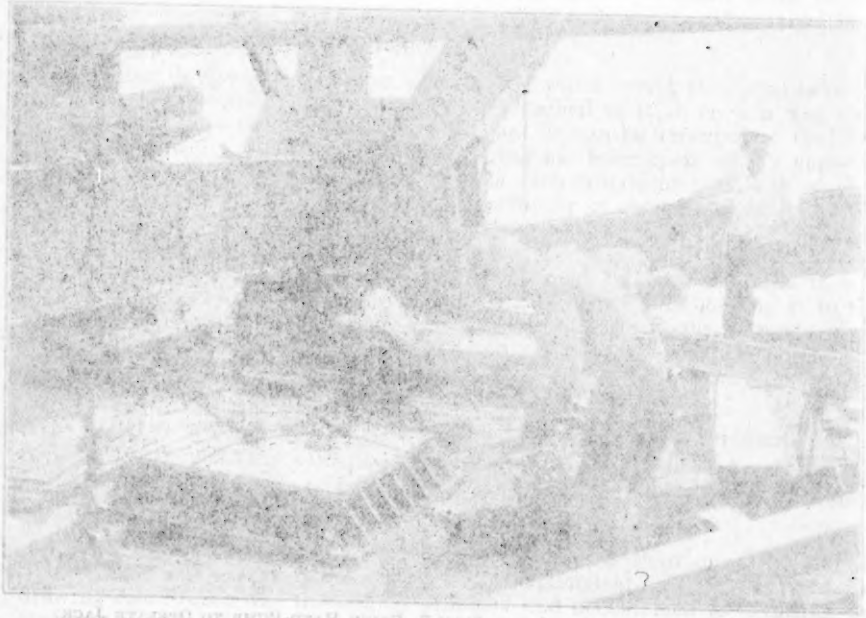


JACK USED FOR WEIGHING REACTION. CASTING AND ROLLER NEST IN PLACE.



RAISING END OF SPAN 6 TO CLOSE SPAN 7, USING HAND-PUMP TO OPERATE JACK.





"Another element of expense in connection with it we found was the making of the many stress calculations which, while they were very interesting, were also very laborious and took a lot of time. On the whole, however, I am inclined to agree with Professor Spofford that that was the only type of structure that could be used in this position, when we consider the joint elements of appearance and economy."

## DEEP PIERS

By George Burrows,* Esq.

Speaking from an intimate acquaintance with the problems connected with fixing the deep piers for the Lake Champlain Bridge, Mr. Burrows gave a condensed, but very interesting account of this part of the work. It bears repetition almost in full:

"On the substructure contract there was nothing out of the ordinary, except perhaps the two deep piers. Our study of the engineers' report on the borings and the soil tests led us to believe that the single-wall coffer-dam could be built in this location. In the design of this coffer-dam the full hydrostatic head was used for that portion above the mud line, and below the mud line a relatively high pressure was used. That is, we assumed about 40 lb. per ft. in weight of the mud. It gave us rather large stresses in the sheeting and in our bracing. The sheeting was designed for a stress of 16 000 lb., and the walls and braces for about 20 000 lb. combined bending and thrust shock.

"It was determined to use the Larson sheeting on account of the interlocking, high-section modulus, and the relatively low weight per square foot of dam. The wells were designed so that the upper portion in the water, the sheeting, was driven close to the lower portion, with an allowance of 6 in. on each side. * * * There was also an additional allowance of about 3 in. between the inside of the wells and the neat line of the concrete, to allow for any irregularities in the wells, or any clearance that might be necessary on account of the sheeting not going as straight as we anticipated.

"The driving of the sheeting was no more difficult, perhaps, than you would expect, considering the skin friction and the tight under-lock, and it was successfully driven down to bed-rock and bedded in the rock.

"The excavation was carried to rock, using a 1-yd. Williams bucket. The cleaning up was done by a sand pump or dredge pump, 10 in., fed by a diver who used a high-pressure hose to scrub or clean off the rock surface.

"The dredging took considerably longer than the concreting. The dredged material was put in dump scows, and dumped in the shallow portions of the lake near-by.

"In concreting we used a plant consisting of a 1-yd. mixer, gasoline driven, with a 55-ton hopper. In addition, there was an extra mixing plant of the same build and capacity held in readiness in case of a breakdown while concreting the deep seal. The concreting was carried on rather slowly; that is, there was no attempt made to break any speed records, and I believe that the results justified our not being too hasty in concreting under water. I think probably about 15 yd. per hour would be what we would consider an average fair day's work in the seal concrete.

"The material was loaded from a dock about 2 miles away, and towed to the job on tug scows, then unloaded into the hopper; and mixed, and placed by underwater buckets which were lowered to the bottom, released, and hoisted

* Engr., Merritt-Chapman and Scott Corporation, New York, N. Y.

again to the top and reloaded. There was but one bucket used at a time in this operation, although several buckets were available in case of emergency.

"The concreting operation was carried on partly in very cold weather, and I believe that the concrete itself was perfectly satisfactory. On pumping out the coffer-dam we found there was very little leakage through the sheeting. After it was pumped out a 4-in. pulsometer would easily hold out any leakage for a depth of 50 ft. of water.

"We owe a great deal to the engineers for their help and co-operation on this job, and I feel that through them and through our very able superintendent the job was very well handled and the results will speak for themselves."

## METHODS FOR BRIDGE FOUNDATIONS

By J. W. Rollins, Jr.,* M. Am. Soc. C. E.

*Double Caissons.*—Foundation engineers, commented Mr. Rollins, would come about as near to agreeing on the proper type of construction for a given job as a group of ladies in the choice of a hat. He admitted that he had never tried such a deep foundation as this with a single line of sheet-piling. In his judgment,

"Any deep foundation like this should be constructed by either the air method, or what is known as the 'open coffer-dam' method; that is, with a structural frame with cells, and operated by dredging through pockets and using the weight in other concrete pockets. That is the old type—I guess the most ancient type of deep foundation."

* * * * * * * * *

"These caissons were 20 by 50 ft. I should have built a shell outside of that, 30 by 60 ft., and braced that sufficiently so that you could use it for the purpose of loading any kind of material in order to sink it. It would have been ideal in this case, because the material was very soft, and I think that caisson could have been properly sunk very easily. That would have left the center all open."

*Dynamite vs. Friction.*—To bolster his argument, Mr. Rollins gave dimensions and mentioned instances covering deep foundation at various sites and under various conditions. One bugbear he mentioned was friction. Enlarging on this topic, he remarked,

"In the matter of friction in coffer-dam construction, of course this depends entirely on the character of material you are going through. At New London we went through sand. We used a very powerful jet there and blew everything out of the way. But at Poughkeepsie we tried all kinds of things on the west caisson to overcome friction. We went down about 100 ft., and had to go 17 ft. more; we would excavate 3 ft. under the cutting-edge, and could not get any motion, although we had tremendous jets scattered all around over the work, with divers working. I think it took a month to sink that caisson 5 ft.

"Finally, we got sort of mad at it, and we put a little dynamite down and gave it a shake. Dynamite is a great worker. You do not want to be around when it is working. But we found that in sinking the other caisson, when we got in any trouble, if we put a charge of dynamite down in the bottom it caused sufficient vibration to get it started. On one side of the river we had great difficulty in getting the caisson through. On the other side we went down

* Pres., Holbrook, Cabot and Rollins Corp., Boston, Mass.

through loose sand, and we never used any dynamite at all. The sand came right to us, and we took it out, and the caisson went down sometimes 2 ft. a day. I think we sank that caisson 25 or 30 ft. in a month. There is the difference in friction. No one would ever believe we would get that east caisson down 135 ft., on account of the trouble with the west one, but we got into that floating sand and it went down beautifully after we got it going."

*Laitance.*—On the subject of laitance, Mr. Rollins was equally positive. In one instance he mentioned, a continuous pouring of 60 ft. of concrete accumulated about 9 ft. of laitance. In another location, it was 20 ft. deep.

"The point is," Mr. Rollins observed, "that when you run against laitance, dig it out, do not let it get set, because it will set so hard that you will think it is concrete; it will set like lime mortar, and you cannot get it out except by using jack-hammers. We found that by taking it out at the time the concrete was deposited, it was soft, and we could take it out with an ordinary bucket, and then clean it up with a jet.

"We filled the pockets at Poughkeepsie about 100 ft. in a continuous operation. That is, we had a day and night force, and I think our best record for 24 hours was 1 200 yd. of concrete."

In general, Mr. Rollins had two suggestions as to improving the resultant concrete. He believes that a relatively shallow pouring each day—say, 4 ft. in depth as at the Lake Champlain Bridge, instead of possibly 20 ft. in some other places he had mentioned—would obviate the difficulty of laitance. Again, in the matter of the size of the container used, he stated,

"My experience in putting in concrete under water is that the concrete is better if you can get it in in large quantities. Two yards at a charge is quite a lot of concrete."

## GENERAL DISCUSSION

*Co-Operation.*—For the general success of this piece of work Frederic H. Fay, M. Am. Soc. C. E., gives greatest credit "to the hearty co-operation of all who have had occasion to take part in the enterprise." In particular, he instanced the matter of foundations and piers. That matter, he observed, was left almost entirely to the contractors. "The proof of the pudding is in the eating" and, therefore, in this case the excellent result justified the means.

In fact, Mr. Fay stated,

"Speaking from an engineering standpoint, if we had this problem presented to us again, I feel quite sure that we would have proceeded in exactly the way that we did in this particular instance. In other words, it is one of those cases where the final carrying out of the project agreed with our earlier conclusions and deductions as to the way it should be done. I am sure, speaking from the standpoint not only of the Engineers, but of the Lake Champlain Bridge Commission—the Interstate body which is in charge of the entire project—that we could not ask for better co-operation or more satisfactory work from contractors than we had in this particular instance."

*Tolls.*—On the economic side, Mr. Fay noted that a very conservative estimate had been made in anticipating the necessary tolls to carry the project. So far were these estimates on the safe side that it appears, at the present rates of toll, the bridge will pay for itself within 15 years instead of 30 years as originally expected. Perhaps, instead, the tolls may be reduced and the period

of payment thereby extended. After that the bridge will be free. This justifies, Mr. Fay believes, the imposing of a moderate toll, and it shows "that the public is well satisfied to pay the moderate toll necessary for the better agency to cross the lake."

*Loads and Stresses.*—In response to a question, Professor Spofford gave the live load for which the bridge was designed as 525 lb. per lin. ft., uniformly, or a concentrated load of 24 000 lb., whichever would cause the greater stress. For the floor construction alone the possible loading was conceived to be two 20-ton trucks side by side in any position.

Regarding the setting of concrete under conditions existing at Lake Champlain, Professor Spofford cited tests of concrete made in cold water, which showed no objectionable effects. Other experiments covered concrete under pressure; these, indeed, showed that the material set more quickly and obtained greater strength than under ordinary conditions.

## RESEARCH IN SOIL MECHANICS AT THE MASSACHUSETTS INSTITUTE OF TECHNOLOGY

By Glennon Gilboy,* Jun. Am. Soc. C. E.

Research in soil mechanics began at the Massachusetts Institute of Technology in the fall of 1925, according to Professor Gilboy. He then traced in broad outline the general treatment of the various developments down to the present. To classify them in a general way, according to Professor Gilboy, these studies,

"* * * fall naturally into two main groups: Research in soil physics, and research in soil engineering. The first group includes studies of the physical properties of soils and of the inter-relation between these properties, with the ultimate object of understanding the nature of the effects observed in soils under various conditions. The second group comprises investigations of the behavior of soil as an engineering material forming part of a structure. The two groups are closely inter-related, inasmuch as a knowledge of the behavior of a soil mass connotes a knowledge of the physical properties of its component parts.

"Problems in soil physics are, in the main, attacked in the laboratory, and include investigations of the compressibility and permeability characteristics, compressive strength, elasticity, internal friction, cohesion, plasticity, hydrodynamic effects, and the influence of grain size, grain shape, uniformity, density, and composition. Investigations of this class form a background for the more elaborate and difficult researches in Soil Engineering, which includes field observations and theoretical developments in addition to laboratory tests."

### Soil Physics

*Mechanical Analysis.*—In passing, Professor Gilboy simply mentioned without detailing the general process of analysis by screening. Another method is that of sedimentation, based on the principle that the rate at which a sphere sinks in a liquid is directly proportional to the diameter of the sphere.

* Asst. Prof. of Soil Mechanics, Mass. Inst. Tech., Cambridge, Mass.

As Professor Gilboy describes this test,

"The apparatus consists of a 2-in. vertical glass tube about 3 ft. long, closed at the bottom, and connected near the bottom through a stop-cock to a small vertical stand-pipe. The sample to be analyzed is mixed up as a dilute suspension in the large tube, the stand-pipe being filled with distilled water. With the stop-cock closed and both tubes stoppered, the apparatus is thoroughly shaken to disperse the soil. It is then hung in a vertical position, the stoppers are removed, and the stop-cock is opened. The suspension in the tube is heavier than water so that the water in the stand-pipe assumes a level higher than that in the tube. The difference in level is a measure of the amount of soil in suspension above the junction of tube and stand-pipe."

The difference in elevation, however, between the two sides of the apparatus is relatively small so that it became necessary to devise a more careful means of measuring. For this purpose the graduated tube in which the elevation was measured, was placed at a very flat inclination, thus extending a relatively small vertical movement over a relatively long measuring distance.

Although this device is quite accurate, its bulk makes it too cumbersome for field use. To meet this objection an attempt has been made to adapt the principle of reading the density by a hydrometer calibrated in some way with the physical characteristics of the soil. This field method is quite rapid and will give results accurate within 10 per cent.

With fine-grained soils, however, mechanical analysis is still far from satisfactory. It appears that,

"* * * flocculation of the colloidal elements of a soil suspension throws serious errors into grain-size determination, and that flocculation is difficult to control or eliminate. Finally, there is no clean-cut relation between the results of a mechanical analysis and the various physical properties of a soil which are of interest to the engineer."

In Professor Gilboy's opinion, however,

"The results of a mechanical analysis should be regarded qualitatively rather than quantitatively. A distribution curve for a soil will unquestionably afford an experienced person an idea of the nature of the material. But his knowledge is far from complete unless supplemented by other tests which yield definite information on those physical properties of the soil affecting the problem in hand."

*Limit Tests.*—An eminent German authority, A. Atterberg, has proposed certain "limit tests" for cohesive materials. These, or an adaptation of them, are now frequently used in America. A sample of clay, for instance, varies in physical characteristics according to its water content; its state may be liquid, plastic, semi-solid, or solid. As the physical changes resulting from varying moisture content are so gradual, it was necessary to set up arbitrary "limits" between these states, expressed in terms of weight of water per unit weight of dry soil.

"The shrinkage limit", states Professor Gilboy, "defines itself and can be determined with comparative ease, * * * from the weight and volume of a sample wet (in the plastic state) and bone dry."

To determine the "plastic limit" a small quantity of wetted material is rolled on a piece of absorbent paper, which causes the water content to decrease

continually. This limit is determined when the sample refuses to form a thread $\frac{1}{8}$ in. in diameter, at which point the moisture content is determined.

The "liquid limit" is not so easy to determine. Because of the inconsistency of the results obtained by various experimenters, a standard method was finally devised using a special apparatus. A part of the material is placed in the bottom of a standardized dished cup attached to the apparatus and by means of a special tool is grooved through the center to a standard shape. By turning a crank the cup is raised 1 cm. and dropped, and the number of blows are counted as required, to close the bottom of the groove over a distance of $\frac{1}{2}$ in. Of course,

"The higher the water content, the fewer the number of blows. A curve of water content against number of blows is plotted, and the water content corresponding to 25 blows, read from the curve, is defined as the liquid limit.

"Limit tests are in somewhat the same category as mechanical analyses, inasmuch as their results have only an indirect bearing upon engineering problems. They are valuable on account of their simplicity in a rapid preliminary grouping of a set of soil samples prior to performing more elaborate tests."

*Permeability.*—The determination of circulation of water through soil is important. According to Darcy's law, the rate of circulation per unit area is directly proportional to the hydraulic gradient. The constant of proportionality, known as the "coefficient of permeability", is the factor to be determined. Of course, the simplest type of apparatus is one which operates under a constant head, from which the coefficient may readily be obtained. This, however, is only suited for relatively permeable materials, such as sand.

"In finer soils, the rate of percolation is so slow that the test occupies an extremely long period of time, and during this time there is the danger that the character of the soil may be altered by organic decomposition and growth of fungi."

This condition led to the development of a variable head permeameter. As the water soaks through the sample the head drops correspondingly and by a reduction formula the constant coefficient can be computed, depending simply on the time interval required for the head to drop a known distance. In such a test, a stop-watch is set, and the test runs itself until the water level lowers a known amount, no matter how long this may take.

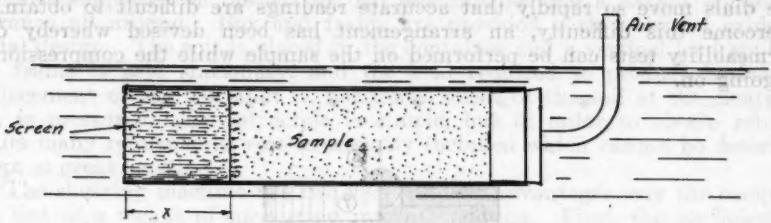
*Capillary Saturation.*—A measure of permeability is also afforded by measuring the capillary rise of water through a sample in a vertical glass tube on the principle that any given material draws up water to a certain definite height. In fine soils, because of the excessive height involved, a long sample is required. However, it is possible to construct master curves showing the rise plotted against time, from which, from a short test, the final results may be predicted.

According to recent studies, however, there are certain inherent defects in this vertical test. For example,

"The original theory* was worked out on the basis of the rise of water in capillary tubes, and applied to soils by substituting an equivalent void diameter for the diameter of the capillary tube. The equivalent or controlling void diameter was assumed constant for the material. As a matter of fact, the

* Terzaghi, "Erdbaumechanik," p. 131.

controlling diameter is the smaller, the higher the rise. For this reason, considerable discrepancies between theoretical and actual curves were observed in extended tests. The errors were minimized by the use of master curves, but the master curves were obtained empirically, so that their applicability was strictly limited to the materials used in the experiments."




TEST FOR CAPILLARY SATURATION.

To overcome some of these objections, saturation in a horizontal direction by capillary action has been attempted. Not only is the theory much simpler, but the actual behavior seems to agree with the theoretical. The test involves the use of a glass tube immersed horizontally in water as shown in the diagram, the water having free access to the sample from one end. According to Professor Gilboy, "this test seems at present to be reliable, and by far the simplest ever devised, especially for rapid field determinations."

*Compressibility and Consolidation.*—In the case of a soil, compressibility is measured by a change in volume under a change in load. Certain phases of this action had previously been discussed in full by Professor Gilboy.*

A coarse material, if saturated, may be compressed quite readily by application of a load; this corresponds to reducing the amount of voids by driving out the water. With a finer material, however, the compression occupies a greater space of time and is known as "consolidation." An elaborate instrument has been devised to determine this factor, as shown more clearly in the diagram.

"The base (1) is bored and channeled to receive a filter disk and allow free water passage. A cylinder (2), 7 cm. in diameter, is fastened to the base by a clamp ring (3), the joint being made water-tight by means of a rubber ring pressed into the joint by a packing ring (4). A close-fitting piston (5) provided with a porous disk slides within the cylinder, transmitting the external pressure to the soil sample. Pressure is applied to the piston through a ball (6) centered by a split plate (7). A pin (8) serves to hold the piston in a definite initial position within the cylinder. When the pin is withdrawn the holes serve as a water overflow. Movement of the piston relative to the cylinder is measured by Ames dials, reading to 0.0001 in., mounted in a dial ring (9) fixed to the cylinder, the stems of the dials bearing against a beam (10) fixed to the piston."

While the material is continuously under water, definite loads are applied and the resulting motion is recorded on the dials. Obviously, as Professor Gilboy explained, the quantity of water forced out by any load, is a gauge of the compressibility of the sample, whereas the rate at which it escapes depends on the permeability. From these considerations it is thus possible to determine the coefficient of permeability.

* "The Compressibility of Sand Mica Mixtures," *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 555.

"True clays", observed Professor Gilboy, "are relatively compressible and impermeable. Their time curves are flat and well defined, permitting accurate computation of the constants. Border-line materials, such as mixtures of very fine sand and silt, are much more permeable and much less compressible than clays. Consequently, the time curves are rather steep in the first stages, and the dials move so rapidly that accurate readings are difficult to obtain. To overcome this difficulty, an arrangement has been devised whereby direct permeability tests can be performed on the sample while the compression test is going on.

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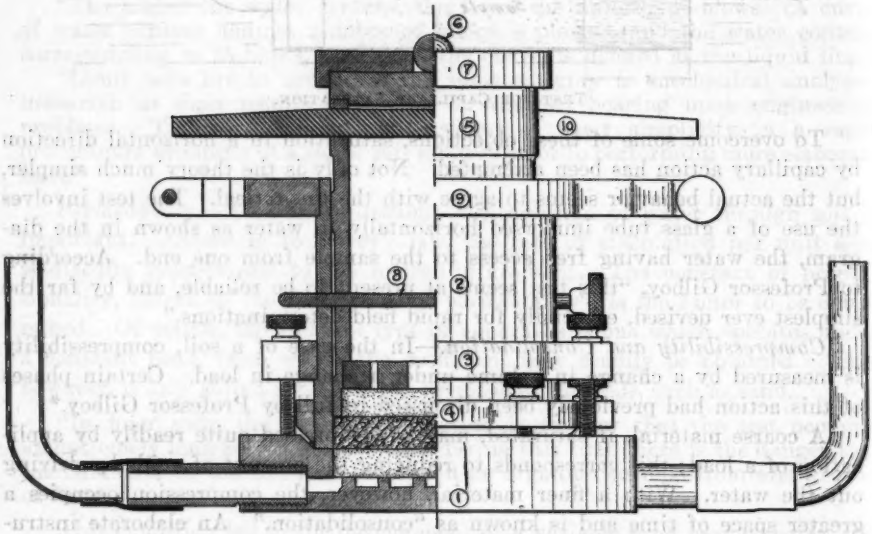



INSTRUMENT FOR TESTING COMPRESSIBILITY.

"The consolidation test is without question one of the most important soil tests ever devised. Its chief advantage lies in the fact that the results are of immediate practical value. An engineer who attempted to estimate the compressive strength of concrete from very elaborate analyses of the stone, sand, and cement, would lay himself open to just criticism, because it is much easier and more reliable to mix up a cylinder and break it in a testing machine. Similarly, the writer does not hesitate to say that a few good consolidation tests on, for example, the core material of a hydraulic-fill dam, will furnish infinitely more information as to the rate of consolidation and the settlement to be expected than a hundred mechanical analyses."

*Shearing Resistance.*—According to Professor Gilboy certain complex phenomena are associated with the resistance of soil, and these he explained in detail. Furthermore,

"* * * a definite intrinsic pressure can and does exist within a cylinder of soil (or any solid body) which retains its shape apparently unaided. If the cylinder is tested in compression, two major effects are noted: First, that there is a definite limit to the pressure the sample will carry; second, that when the limiting pressure—the compressive strength—is reached, the sample fails in shear along certain well-defined planes, whose traces appear as cracks in the surface."

To study these and similar features, an elaborate machine has recently been devised which he described as follows:

"The nucleus of the device is a rigid rectangular brass container consisting of two parts, a box resting on a movable platform, and a frame of the same size supported on the box by ball bearings, leaving a narrow horizontal clearance all around. Box and frame are provided with tool-steel grids of special design. The soil is placed in the container and a vertical load applied. The frame is held stationary, and the load required to produce horizontal displacement of the box, that is, horizontal shear of the soil at the clearance line, is measured. The principle is simple, but in order to obtain reliable results many refinements were necessarily included which cannot be described except at great length.

"The shearing machine has two very distinct advantages over the compression test as a means of measuring internal friction. First, the coefficient of friction is measured directly; second, the machine is applicable to all types of soils. The first series of tests, now nearing completion, has yielded very interesting information. To quote only one example, the tests have shown that current ideas relating to the internal friction of sands will have to be revised. The angle of internal friction of dense sands is by no means constant, but decreases rapidly under increasing surface pressure. Angles of friction of 60° in dense sand are not uncommon."

## Soil Engineering

The practical application of these principles to solve various problems arising in construction comprise a most interesting part of this work. Professor Gilboy listed a few of the projects:

"The experiments performed on the borrow-pit materials for a proposed hydraulic-fill dam in Pennsylvania furnished the data required for preparing a suitable program for the investigation of the core of the Germantown Dam [Miami Conservancy District]. The soil studies for two large earth dams in Massachusetts called attention to the shortcomings of the methods for computing the permeability from the results of mechanical analyses, and provided an incentive for developing rapid methods for direct permeability tests. An extensive ground-water survey in the flats along the Connecticut River furnished for the first time conclusive information on the air content of alluvial deposits. Test borings in Arkansas and drainage investigations in Spanish Honduras furnished data on the permeability of stiff clay soils with crumbly structure."

*Bearing Capacity.*—In one instance a group of buildings founded upon piles was observed to be settling at an alarming rate. As the piles themselves seemed to be sufficiently strong, the conclusion was unavoidable that a compressible layer of soil far beneath was in the course of consolidation.

Accordingly, consolidation tests were performed on this particular clay to determine its compressibility and permeability characteristics. Following this, a tremendous amount of computation was necessary to determine the settlement which might rationally take place. "The agreement between the theoretical settlements and those actually observed at the site was remarkable." The striking thing about this investigation was the fact that the action might have been anticipated before the construction proper. This in itself is proof of the value of soil mechanics in practical engineering.

"Progress in this line would be greatly facilitated if every construction job were looked upon as a large-scale experiment. The data required are com-

paratively easy to collect. A load-progress diagram, a series of bench-mark observations, a geological profile of the underground, and a few soil tests on the different strata, will furnish the information required for an analysis of the causes, rates, and ultimate amounts of settlement. The additional cost of such observations would be a negligible item in the total cost of construction; the information obtained would be of highest value."

*Frost Heaving of Highways.*—In this work,

"The experimental unit consisted of a wooden box, 3 ft. square and 3 ft. deep. On one side of the box was an observation well 1 ft. square, communicating with the interior of the box at the bottom. A gravel filter was placed in the bottom of the box and well, thick enough to cover the opening between them. In this manner, the water level within the box could be controlled from the well, the gravel preventing any soil from entering the well. The sides of the box were coated on the inside with a thin layer of grease, over which were laid small overlapping sheets of Manila wax paper. The object of the grease and paper was to minimize the retarding effect of friction between the soil and the sides of the box. The soil to be investigated was carefully tamped into the box, filling it level full. On top was placed a slab, 2 ft. 8 in. square, of concrete or macadam, representing the road surface."

Eight units of this sort were buried in soil during an entire winter. In spite of some difficulties, valuable results were obtained:

"The researches showed that frost heaving cannot occur to any appreciable extent unless there is a continuous supply of water from below the frost level. The pressures associated with freezing water are due to the growth of ice crystals in layers, this growth being in turn due to travel of the large trihydrol molecules, $(H_2O)_3$, toward the focus of crystallization. As long as trihydrols are supplied, crystal formation will continue, and pressure will be exerted to lift the soil above the ice layer. If the supply of trihydrols is cut off, equilibrium will be established and the heaving will cease.

"It has been observed that ice layers will grow in mixed fine-grained soils, while the water in the voids of clean sands will freeze without causing growth of the crystals beyond the ordinary increase in volume associated with the change in state. This behavior is due to the fact that water in very small spaces exhibits the property of remaining in a liquid state at temperatures considerably below the normal freezing point. Since the freezing temperature of water in very fine voids decreases with decreasing void diameter, it can be seen that in the larger voids the water will crystallize while in the surrounding smaller voids the water will remain in a liquid state, thus keeping supply channels open for the growth of the ice crystals.

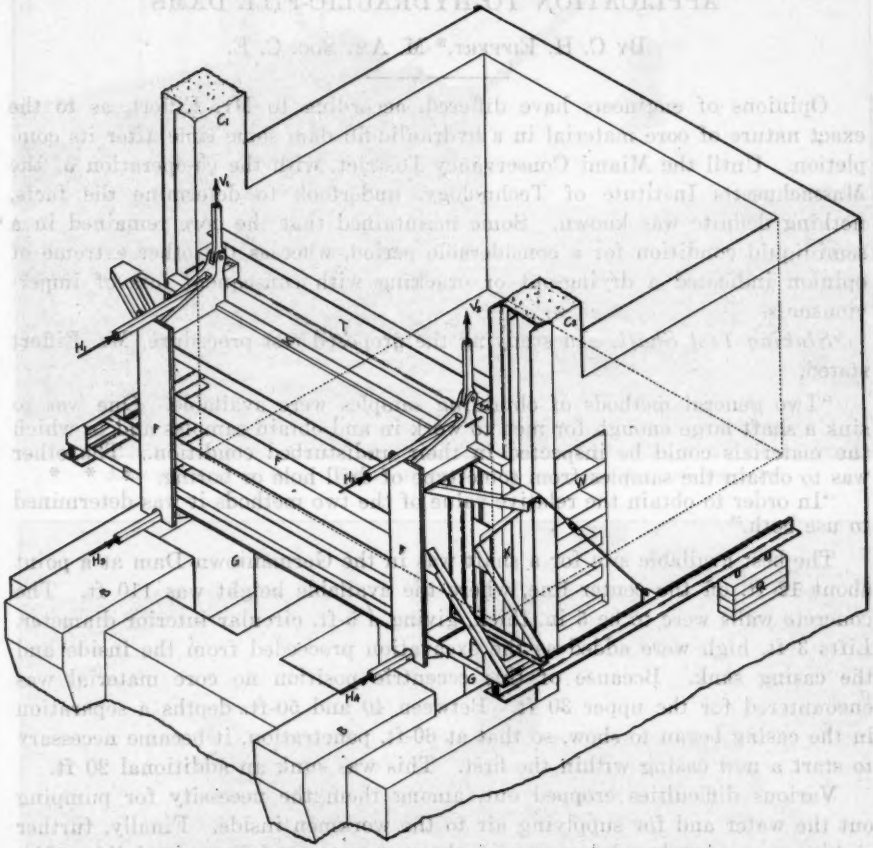
"As the frost effect penetrates deeper into the soil mass, another ice layer will start to form lower down. As the lower layer grows, it gradually cuts off the water supply, so that further growth of the upper layer is prevented.

"The frost effect is thus shown to be controlled by three factors: First, the position of the ground-water level, or of veins and pockets of water which may serve as a source of supply for excess ice accumulation; second, the void-size distribution of the soil in which freezing occurs; and, third, the capillary properties of the soil between the source of water supply and the frost level."

*Earth Pressure on Retaining Walls.*—The latest and most elaborate project, according to Professor Gilboy, is that on lateral earth pressures exerted behind a retaining wall, as measured on a large-scale experimental apparatus shown in the diagram in isometric projection. He described this in detail showing how the various measurements and pressures could be obtained, eliminating the effects of outside structures or forces. To illustrate the delicacy of the

apparatus it may be noted that wall movements are controlled to within 0.0001 in.

Knowing the vertical and horizontal components at various places (marked *V* and *H* on the drawing), it becomes possible to determine the magnitude, direction, and point of application of the earth pressure. Professor Gilboy thus summarized the value and scope of this important apparatus:




APPARATUS FOR DETERMINING SHEARING RESISTANCE OF SOILS.

"In the design of the apparatus, an attempt was made to eliminate the shortcomings of previous installations of similar character, particular attention being given to the rigidity of the retaining wall and the possibility of moving the wall at will through an appreciable distance from the bin. The distinguishing features of the Institute apparatus are as follows: The model wall is practically rigid, deflections under maximum pressure being less than 0.001 in.; the wall can be tipped or moved parallel to itself through a distance of more than an inch; and the joint between bin and wall is equipped with a water-tight rubber bellows or diaphragm so that the back-fill can be completely saturated without any leakage of water.

* * * * * *

"Surcharges up to 50 tons can be applied to the surface of the back-fill. The effects of different slopes of the surface of the back-fill can be measured. The action of filters and drains, as well as the effect of partial and com-

plete saturation can be studied. The apparatus is just starting in on a lifetime of valuable service, and will unquestionably furnish interesting and reliable information on many of the difficult problems connected with retaining walls."

## DISCUSSION

### APPLICATION TO HYDRAULIC-FILL DAMS

By C. H. Eiffert,* M. Am. Soc. C. E.

Opinions of engineers have differed, according to Mr. Eiffert, as to the exact nature of core material in a hydraulic-fill dam some time after its completion. Until the Miami Conservancy District, with the co-operation of the Massachusetts Institute of Technology, undertook to determine the facts, nothing definite was known. Some maintained that the core remained in a semi-liquid condition for a considerable period, whereas the other extreme of opinion indicated a drying-out or cracking with consequent loss of imperviousness.

*Sinking Test Shaft.*—In studying the proposed test procedure, Mr. Eiffert stated,

"Two general methods of obtaining samples were available. One was to sink a shaft large enough for men to work in and obtain samples and in which the materials could be inspected in their undisturbed condition. The other was to obtain the samples from some type of drill hole or boring. * * *

"In order to obtain the relative value of the two methods it was determined to use both."

The best available site for a shaft was in the Germantown Dam at a point about 12 ft. off the center line, where the available height was 110 ft. The concrete walls were to be 5 in. thick, giving a 5-ft. circular interior diameter. Lifts 3 ft. high were added as the excavation proceeded from the inside and the casing sank. Because of this eccentric position no core material was encountered for the upper 30 ft. Between 40 and 50-ft. depths a separation in the casing began to show, so that at 60-ft. penetration, it became necessary to start a new casing within the first. This was sunk an additional 20 ft.

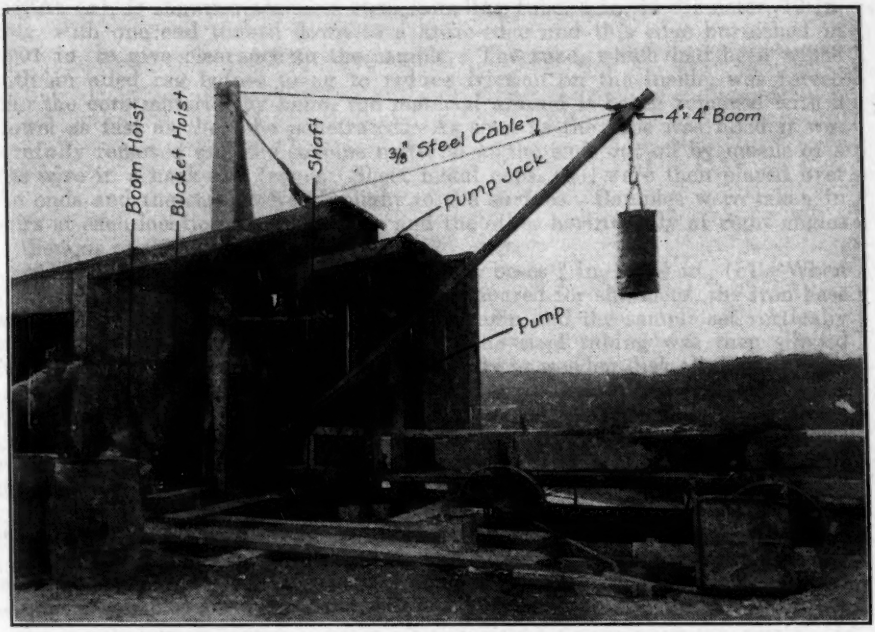
Various difficulties cropped out, among them the necessity for pumping out the water and for supplying air to the workmen inside. Finally, further sinking was abandoned because of slow progress and hazard of life. Mr. Eiffert believes that,

"If another shaft were to be sunk in material similar to that at Germantown, it could be done very satisfactorily by the methods described but that the walls should be 8 in. thick to give them additional weight for sinking and they should be reinforced to prevent any separation at the construction joints."

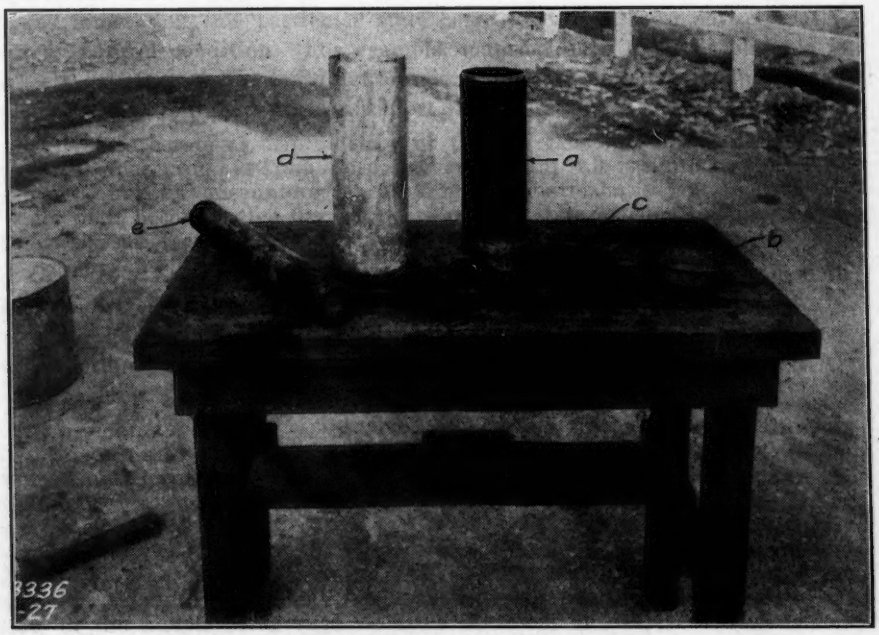
Samples were taken in pairs at right angles, the first at a depth of 33 ft. It was not practicable to secure additional samples every 10 ft., as anticipated, so that in all only four sets of good samples were obtained.

*Sampling Apparatus.*—Various parts of the sampling apparatus are shown in the accompanying photographs. This apparatus is described by Mr. Eiffert as follows:

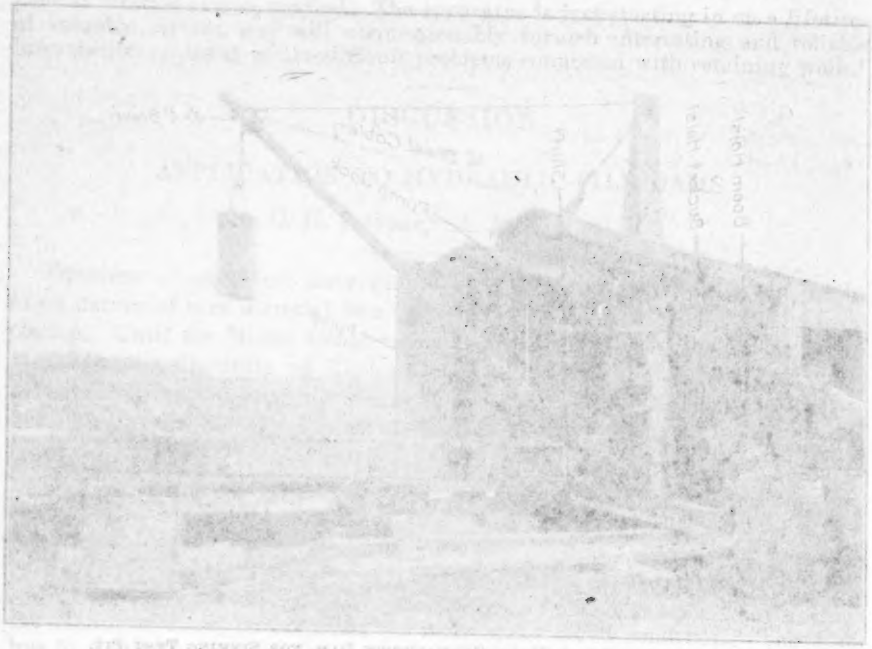
* Chf. Engr., The Miami Conservancy Dist., Dayton, Ohio.

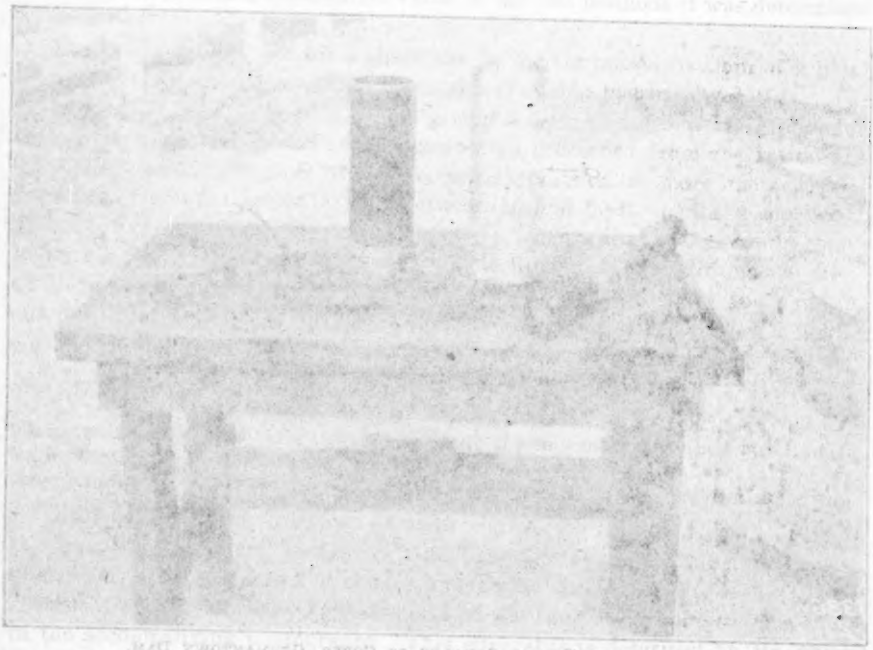



ARRANGEMENT OF SHAFT HEAD, GERMANTOWN DAM, FOR SINKING TEST PIT.




APPARATUS FOR TAKING SAMPLES OF CORES, GERMANTOWN DAM.





"At (*a*), is shown a seamless steel sampling tube, 4 in. in diameter, 12 in. long, with one end turned down to a knife-edge and this edge burnished in 0.001 in. to give clearance to the sample. The tube, which had been wiped with an oiled rag before using to reduce friction on the inside, was forced into the core material by hand, the material around it being removed with a trowel as fast as the tube penetrated. As soon as the tube was filled it was carefully removed and the surplus material at the ends cut off by means of a fine wire in a hack-saw frame. Sheet metal caps, (*b*), were then placed over the ends and the sample was brought to the surface. Samples were taken in pairs at each location, one vertically and the other horizontally at right angles to the axis of the dam.

"On the work table were two circular iron bases $\frac{3}{4}$ in. by $4\frac{1}{2}$ in., (*c*). When a sample was brought to the surface to be prepared for shipment, the iron base was covered with a thin layer of melted paraffin and the sample set vertically on it after the caps had been removed. The steel tubing was then slipped from the sample. If it did not come off easily, a wooden disk slightly smaller in diameter than the tube was placed on top of the sample and held down while the tubing was pulled up off the sample. A cylinder, (*d*), of 24-gauge, galvanized iron, $4\frac{1}{2}$ in. in diameter and $13\frac{1}{2}$ in. long, was then placed over the sample, the bottom sealed with modeling clay, and the space around and above the sample filled with melted paraffin. When the paraffin had hardened the cylinder was reversed, heat being applied to the iron disk if necessary to loosen it, and the other end poured full of the paraffin. The entire sample was thus enclosed in paraffin, the structure of the sample and its water content being well preserved until it arrived at the laboratory. The tube, (*e*), was used for obtaining samples from the drill hole."

*Results of Tests.*—Upon examination in the laboratory, the material was reported as "25% consolidated." At first, this was somewhat alarming, until it was found that this particular state corresponded with an entirely satisfactory natural condition. For example, material only 10% consolidated is entirely satisfactory for building foundations in Boston. Referring to the general results of these tests, Mr. Eiffert stated that,

"The future rate of consolidation of the Germantown core is necessarily somewhat uncertain, but from a practical standpoint this is unimportant. The quantity of water encountered in sinking the shaft shows that there is no danger of it becoming too dry while its present state of consolidation makes it entirely satisfactory for core purposes."

*Drill-Hole Tests.*—To substantiate the finding from the test pits, a drill hole was driven on the opposite side of the center line of the dam about 25 ft. from the shaft, using a pipe $5\frac{5}{8}$ in. in diameter. At various points all the way to the bottom of the dam, samples were taken.

Describing this work, Mr. Eiffert continued,

"The sampling tools provided consisted of pieces of threaded 2-in. seamless steel pipe, 24 in. long (shown in the photograph at (*e*)). The rod for lowering these samplers consisted of $1\frac{1}{4}$-in. pipe to the lower end of which was attached a piece of 2-in. pipe about 2 ft. long. When a sample was desired the sample tube was screwed into the lower end of the 2-in. pipe. After the well had been carefully cleaned out with a dipper the sampling tool was lowered and driven into the core about 3 ft. This insured the complete filling of the sampling tube. The samples remained in these tubes for shipment, about $\frac{3}{4}$ in. of the sample being removed from each end and replaced with melted paraffin."

In all, seven satisfactory samples were obtained throughout the depth of the dam. While detailed results are not yet available, a comparison of the

two methods enables the conclusion that "a good drill hole sample will furnish results as reliable and consistent as the undisturbed sample from the shaft." All in all, Mr. Eiffert is enthusiastic in his belief that,

"The studies of core consolidation, permeability, and the graphical method of determining the stability of the shell material during construction should be of great practical value to engineers designing hydraulic-fill dams."

## GENERAL DISCUSSION

### SOIL RESEARCH

*Assistance for Research.*—Speaking on the subject of the business side of these researches, C. M. Spofford, M. Am. Soc. C. E., observed that it was not entirely easy for an institution of education to provide the necessary wherewithal. One of its principal functions, he asserted, was to train young men capable of continuing the work.

In addition, Professor Spofford acknowledged the warm spirit of co-operation exhibited by outside organizations such as the U. S. Bureau of Public Roads, the Miami Conservancy District, and the New England Power Company. It is reasonable to expect that such co-operation and the worth-while results of investigation will continue.

*Mile-Posts in Foundation Research.*—To George Paaswell, M. Am. Soc. C. E., it appeared that the researches outlined marked most definite departure beyond and above anything else of a similar nature that has been done in the United States. Regarding the future of the work, he was quite enthusiastic. These studies having at last provided a means of classifying the soils, he asserted,

"We are now in a position to make a really rational study of the foundation structure. We can identify our soils by means of a positive analysis—no longer by the grain count or the grain size, but apparently by the behavior of the soil itself, which is based upon definite properties, you might say, analogous to the properties we have in our structural materials, such as concrete and steel. We can design a foundation that will almost without question behave as we expect it to behave, if we apply these results."

The same could be applied further to construction processes dealing with soils, whether referring to the type of machinery to use, the proper sheeting and bracing, or the necessary measures of underpinning. The entire success of the investigation, Mr. Paaswell believes, is a magnificent tribute to the ability of Charles Terzaghi, M. Am. Soc. C. E., who originated and carried on much of the work.

*Boussinesq's Formula.*—Answering a question regarding the dispersion of pressure diagonally through adjacent soil, Professor Gilboy stated that the best computations could be made by the so-called Boussinesq theory. This assumes that the material is homogeneous and, in this respect, of course, is not exact; however, it is the nearest approximation that can be made and gives results that are close to the actual findings. The main objection to the formula is that it is rather involved and laborious in application. The great value of this formula was warmly attested by Anson Marston, President, Am. Soc. C. E., who reported having successfully used it on a problem of pressure transmission through soil.

# POWER DIVISION

OCTOBER 10, 1929—2:15 P. M. TO 5:30 P. M.

## KAPLAN TURBINES AT THE RYBURG-SCHWORSTADT PLANT ON THE UPPER RHINE

By Dr. Ing. D. Thoma*

To describe this interesting plant Dr. Thoma used numerous slides. Running comments on these practically constituted his paper. As Professor of Hydraulics, Water Power Machinery, and Water Power Plants, and as Director of the Hydraulic Institute of the Technische Hochschule at Munich, Germany, he had gained a first-hand knowledge of his subject.

*General Features.*—Dr. Thoma described the setting of the plant and its location, as follows:

"The Ryburg-Schworstadt water power plant is located on the Upper Rhine between the State of Baden, Germany, and Switzerland. The frontier between the two countries is just in the middle of the river. The plant is being built by a corporation of four members, consisting of one company controlled by the State of Baden, one company controlled by the Swiss Government, and two private companies, one German and one Swiss.

"The flow of the river is, for 50% of the time, equal or greater than 35 000 cu. ft. per sec. In time of low water, the flow is only about one-half this figure. These favorable conditions regarding the flow are due to the fact that on the upper stretch of the river is a great lake, the Bodensee.

"The river banks on the upper side of the dam are quite high, and have allowed the raising of the head-water level to the extent of about 40 ft., and only a very few levees had to be built in order to prevent the flooding of the land.

"It was decided to install four turbines, so to speak, one for each of the four members of the company. The turbines are installed on the right side of the dam, facing down stream. On the left side of the dam are installed the spillway gates. In order to maintain full power in times of flood the turbines will have a maximum discharge of 11 000 cu. ft. per sec. for each unit. The turbines of course had to be very big, the runner diameter being 23 ft."

*Choice of Propeller Turbines.*—Except at one place, where there is a ravine extending about 60 ft. below the normal level, the river bed was solid rock, and the plant and turbines thus had good foundations. The ravine was properly filled in to supply the necessary foundation for one of the turbines.

The units will possibly have to be operated separately, and it is desirable, therefore, that the turbines have high efficiencies even at small loads. On the

* Prof. of Hydraulics, Water Power Machinery, and Water Power Plants, and Director, Hydr. Inst., Technische Hochschule, Munich, Germany; Visiting Prof. of Hydraulics, Mass. Inst. Tech., Cambridge, Mass.

other hand, they were required to operate under low head so that the specific speed had to be as great as possible. These considerations led to the selection of propeller turbines with movable runner vanes—that is, Kaplan turbines.

The units installed were designed for a head of 39.5 ft. and a maximum discharge of 11 000 cu. ft. per sec. Their outputs were 42 000 h.p. each at 75 rev. per sec.

*Steps in the Development of the Kaplan Turbine.*—Dr. Thoma showed views of a series of runners tested by Dr. Kaplan about 1912. The tests were on a small scale, with runners 6 in. in diameter. The blades were of sheet copper and could be bent to any shape or position so that different experiments could be made.

The next step was to investigate the form of draft-tube best suited for this new form of turbine. Dr. Kaplan first used straight glass tubes so that he could see the conditions of flow. Later, he worked with bent draft-tubes, made of sheet metal.

One difficulty encountered in the constructional development was in connection with the mechanism by which the blades and pivots can be rotated about their axes through the levers on the pivots connected with a central rod within the interior of the shaft. Dr. Thoma showed three steps in the evolution of an arrangement that would solve this problem. Further difficulty was experienced in connecting the governor with the rotating system of the turbine.

*Advantages and Efficiencies.*—Commenting on these difficulties, Dr. Thoma stated that:

"Of course, the whole design and construction of a turbine of this kind is more complicated than an ordinary turbine with fixed runner vanes; and to justify the adoption of this design there must be advantages in the use of this type of turbine.

"The advantage of the Kaplan turbine is the combination of a specific speed as high as that of the propeller turbine with fixed blades, with an efficiency curve which is flatter than even the efficiency curve of the propeller turbine for a low specific speed."

* * * * * *

"The Kaplan turbine will always be more expensive than an ordinary turbine, but the additional cost will in general be only a small percentage of the whole cost of the power plant, so that if, by the adoption of movable runner blades, the whole power plant is made more useful the additional expense in many cases will be warranted."

Acceptance tests made on Kaplan turbines have shown efficiencies as high as 95 per cent. For smaller plants of about 300 h.p., of course, the efficiencies will be lower.

*Gates.*—At the Ryburg-Schworstadt Plant, the effect of cavitation was a serious problem. With constant speed, position of guide-vanes and gate-opening, cavitation can be observed with an increase in suction head. Numerous laboratory tests on cavitation were made especially for this plant.

The designers of the plan decided against the use of intake gates because the turbines can be stopped either by closing the guide-vanes or by moving the blades of the runner. When repairs are necessary, stop-logs can be used and,

therefore, the intake gates were omitted. This arrangement indicates a large degree of faith in the reliability of the turbines.

*The Turbine.*—In order to guard against cavitation, five runner blades were used instead of the customary four. They are made of cast steel, with 2% nickel. Concerning their arrangement, Dr. Thoma stated that the total surface of the blades must be large. Therefore,

"The outer wall in the region below the axis of the pivots of the blades has the form of a sphere. In this way the gap between the blades and the outer wall is maintained the same at all angles of the blades. In the upper region the outer wall is cylindrical in order to facilitate the insertion of the runner. Of course, the gap at the tip of the blades must be made so that with a small angle the blades will not touch the outer wall; with larger angles the gap will be increased.

"Experiments have shown that the effect of this increasing gap is less on the upper side, in the region of the entrance edge, than it is in the region near the exit edge, and that is why it seemed permissible to make the upper part of the wall cylindrical. The hub, too, has a spherical form in the region of the blades, for the same reason."

*The Kaplan Turbine in Sweden.*—A project now being planned in Sweden contemplates the use of a Kaplan turbine about 26 ft. in diameter. It is to have a discharge of about 9 000 cu. ft. per sec., with an average head of about 12 ft. As in the case of the Ryburg-Schworstadt Plant, these Swedish turbines will have no intake gates; but the elevation of the guide-wheel will be increased so that, by admitting air to the intake, the flow can be stopped. In conclusion, Dr. Thoma declared that the Kaplan turbine is now in general use in Europe, but,

"It is not proposed that all plants, or even all low-head plants, should be fitted with Kaplan turbines. The propeller turbines with fixed blades, for the splendid development of which every credit must be given to American engineers, will retain a field of useful application. But on the other hand there are many low-head power sites in the development of which the Kaplan turbine, either alone or in combination with turbines with fixed blades, would secure an improvement in economical operating conditions."

## DISCUSSION

In response to questions by Chairman L. J. Bevan, Messrs. H. J. Burrows, R. E. B. Sharp, E. G. Lee, B. F. Groat, H. W. Dennis, E. A. Dow, and others, Dr. Thoma elaborated his subject further in discussion. For instance, it was stated that the weight of the blades constituted about 50% of the entire weight of the turbine. The weights on this type are about the same as those used on the Francis wheel.

Speaking of efficiencies, it was shown that a smaller type of Kaplan wheel installed in Sweden had an efficiency of more than 80% from about 25% of the full load point. This is an unusually long range and a more constant efficiency than even an impulse wheel will show. The propeller type without the movable blades has an exceedingly shorter range.

While the water can be shut off by means of the movable vanes there is no way to reverse the direction of rotation. If the guide-vanes are closed also, the turbine can be stopped by the runner-blades, but the discharge will not be stopped.

*Movable Blades on Boat Propeller.*—Many marine propellers with movable blades have been designed, but the purpose has not been so much to adjust the pitch to the load, as to reverse the load without reversing the direction of rotation of the shaft. Dr. Thoma expressed his belief that the movable blade would be applicable to the problem of towing different loads so as to get more economy out of a tugboat. The only question would be whether it would pay to install it.

*Angular Movement of Blades.*—Between the position of full load and no load the total angular movement on the axis of any one blade may be, roughly, 30°, according to Dr. Thoma. Attention was called to the fact the rate of movement of the runner vanes is slower than that of the guide-vanes. They all begin simultaneously, but the runner vanes do not reach their final position until several seconds later than the others. Furthermore,

"It has been proposed to slow down when the load increases, to have the movement of the runner vanes quite slow in order, first, not to have too large a change of discharge, but only enough efficiency to prevent water and other disturbances, gradually adjusting the runner vanes to the correct angle. I do not know whether the effect would be very great, but it is possible to make it so that on an increase of the load first several guide-vanes are closed, the runner vanes retaining the same position. Then the power of the turbine will decrease without the discharge decreasing very much, and the runner blades are gradually adjusted to a smaller angle, and the discharge is also decreased, but very slowly. That can be done.

"Of course, with the increase of the load the positions should be moved simultaneously, because it is desired to have as much power as possible with the increase of load, and they can be arranged so they will increase only with the proper flood. The runner vanes can be made so that they are closed later than the guide-vanes."

He explained at length the reason for five blades instead of four. In Professor Thoma's own words:

"In order to reduce the danger of cavitation or the liability to cavitation, the total area of the blades should be not too small. You need a relatively large area for the blades. If you incorporate this area in four blades, the blades will, of course, come out longer than with five blades or six blades. That is, when the blades are adjusted to different angles, the gaps which result between the cylindrical part of the outer wall (that is, the upper part) and the blades will be greater if we have fewer blades. If you had ten blades, for instance, the gaps would be very small. If you had only four blades the gaps would be greater, so it is better to have many blades. But with six blades the design of the hub was too difficult. The parts could not be designed so that all would fit easily in the hub. It could have been done, but with some difficulties, and it was decided that it was better to have five blades."

*Characteristic Curves.*—Curves were presented to show that, as the draft head increased, there appeared a critical point at which cavitation began. At that point there was a sudden drop in efficiency and a decrease in discharge.

Whether the discharge will decrease or increase at this point will depend on the shape of the blocks.

"When cavitation begins the whole flow is changed" said Dr. Thoma. "It is somewhat similar to the effect upon an airplane. The coefficient of uplift suddenly goes down if a certain angle is exceeded. * * * There are certain critical angles at which the lifting force decreases, because the flow is changed quite radically, and that is what happens at the beginning of cavitation, with certain shapes of blades. Before reaching the critical point the flow follows the blade, and when the critical point is reached, and cavitation begins, it has another effect; on the suction side the flow does not follow the flow of the blade, but leaves it. This is not necessary. The formation of back-water space is not necessarily connected with cavitation."

*Inclined Guide-Vanes.*—If the mechanical difficulty could be overcome, will there be an advantage in having guide-vanes inclined instead of parallel? Professor Thoma felt that there would be an advantage, but the mechanical design is not simple.

# HIGHWAY DIVISION

OCTOBER 10, 1929—2:15 P. M. TO 5:15 P. M.

## EQUITABLE DISTRIBUTION FOR HIGHWAY PURPOSES OF MOTOR VEHICLE LICENSE FEES AND GASOLINE TAXES

By George H. Henderson,* M. Am. Soc. C. E.

Opinions as to distribution of income from automobiles vary considerably in different parts of the country. Mr. Henderson gave a comprehensive review of these differences. Some believe that registration and gasoline tax fees should be collected, and spent, by the State for improving rural roads under the supervision of a State highway department or commission.

"In many States", said Mr. Henderson, "it is the opinion that certain portions of the fees collected should be turned over to the county organizations to be spent for the improvement of roads within the county under the supervision of the county officials. In a few States it is considered proper to transfer portions of the fees fund to cities and towns for use on city and town roads. In several States part of the fees collected are spent to defray carrying charges on outstanding road and bridge bonds. In at least two cases all or part of the gasoline tax fees collected go into the general fund of the State and are used for the general purposes of the State, while in three States part of the fees are devoted to educational purposes.

*"The Fundamental Question."*—Doubtless the varying opinions on the subject are influenced by local conditions. Eliminating the opinions of the very small minority who believe that a portion of the registration and gas tax fees should be expended for educational purposes, we find an almost unanimous opinion that the funds derived from the above fees should be expended only for highway purposes. If now we eliminate all questions as to the percentage or proportion of the funds derived from the registration fees and gasoline tax fees which should be expended for maintenance and reconstruction of roads or the construction of new roads, or the amount of money to be set aside for interest and sinking funds on outstanding highway and bridge bonds, costs of collecting the above fees and other logical administration charges, it then becomes a question, of where the money shall be spent and who shall spend it.

*"State's Jurisdiction Over Improvements."*—On the supposition that registration fees and gasoline fees are assessed by the State as a State-wide tax, it is the opinion of the writer that the money should be expended to benefit the State as a whole and that this cannot be accomplished as well if the funds are spent by local sub-divisions of the State as it could be if spent by a State organization, such as a State highway commission or department. Regardless of where the money is spent, the State department should have full jurisdiction in the selection of the improvements to be made, and its approval also should be necessary as to the method of improvements. In this way, the

* Chf. Engr., State Board of Public Roads, Rumford, R. I.

writer believes, a system of roads can be built up within the State which will interlock to form a satisfactory system of main trunk roads, secondary roads, and feeder roads."

*Rural Roads Connecting Cities.*—At first the hard-surfaced roads were confined to cities; but even before motor vehicles appeared, good communicating roads were being built between centers of population. These roads were mostly financed by towns, counties, and the State from general funds. Registration fees and gasoline tax fees later provided additional funds.

In the words of Mr. Henderson,

"Until comparatively recently there was little question but that the funds derived from these sources should be spent on such rural roads as would best serve the interests of the State as a whole. Now, in some sections of the country, one occasionally hears the opinion expressed that the cities should receive a portion of the fees collected for the improvement of streets or roads within the city limits, or that the fees fund should be spent in the political sub-divisions of the State in accordance with some plan that would take into account the proportion of the total fees contributed by motor vehicles taxed within such political sub-divisions. Among the arguments set forth by those who believe the cities should receive a share of these fees is that most of the traffic in the cities is local and does not use State and county highways to any extent, that many vehicles are burdened with a gasoline tax which are hardly, if ever, operated on rural highways. The cities also complain that they receive no benefit from motor vehicle taxes to offset added expense incurred in traffic supervision, safety devices, and in constructing and maintaining more expensive pavements to bear an ever-increasing volume of traffic."

*Who Pays?*—While there is doubtless some merit in such arguments, Mr. Henderson emphasized the fact that money for highway improvements must be collected where the money is. Most automobiles are owned within cities and, therefore, this group must contribute. Speaking of spending State funds on city streets, Mr. Henderson declared,

"It may be admitted that if the improvements on State highways or rural roads have reached the stage where further improvements to facilitate traffic (mostly city-owned vehicles) are not needed or justified, then a share of the fees fund may logically be spent within the limits of the cities. On the other hand, if the funds derived from registration, license, and gas tax fees are no more than sufficient to carry out logical and needed improvements, together with the maintenance of roads outside the cities, then it seems it would be unwise to divert any portion of the funds available for use within the cities, because in this event it logically would follow that additional funds would have to be provided for these needed improvements outside the cities, which funds would probably be obtained through an additional motor vehicle fee, or gas fee, or both, the burden of which would be borne for the most part by city-owned vehicles.

"*Taxes on Real Property.*—It is sometimes said that the ease with which motor vehicle and gas tax fees are assessed and collected prompts certain interests to advocate the further burdening of the motor vehicle owner in order to relieve the general taxpayer. While motor vehicle fees and gasoline tax fees are a legitimate and easy method of deriving funds for highway improvements, the assessment of these taxes alone for all forms of highway improvement, including new construction, is not strictly justified, even though we realize that one out of six or seven persons to-day owns a motor vehicle and derives direct benefit from the construction and improvement of highways. At least a share of the cost of construction of new roads—a capital

investment—should be borne by real property inasmuch as such construction, generally speaking, benefits real property. We all are aware of the manner in which property valuations soar in the rural districts upon the completion of a modern pavement on a State highway and sometimes even in anticipation of such construction. This enhancement of rural property is a direct benefit to the property owners, some of whom do not own motor vehicles, and this enhancement of the property usually results in higher valuations and increased taxes for the towns in which the property is located; and the argument holds equally true in the case of the construction of new streets within the limits of cities."

*City Traffic on Rural Roads.*—Three New England States conducted traffic surveys and discovered that 90% of the traffic on rural roads was by city-owned vehicles; of commercial vehicles, 80% were city-owned. Experience in Ohio agrees with this result and supports the contention that a large portion of the gasoline used by city-owned motor vehicles is consumed outside the cities.

According to Mr. Henderson,

"The demand for wider roads, parallel roads, by-passes, grade-crossing eliminations, etc., is constantly increasing because of the constantly increasing traffic, most of which originates in the cities. The construction and improvement of State roads has extended the cities' business to the limits of these roads. Comparatively few years ago large business houses in certain cities confined their merchandise deliveries to within the city limits or the suburban areas immediately surrounding the city. To-day, these same business houses are doing business and delivering goods all over the State. If city-owned vehicles were eliminated from rural highways, the enormous amount of money being spent for modern pavements, wider pavements, parallel roads, by-passes, grade-crossing eliminations, etc., could be greatly reduced, because without the city-owned vehicles the traffic on the rural roads would not warrant such expenditures.

*"The Commuter.*—With the modern roads of to-day we find certain citizens maintaining country homes for the summer months, in addition to their year-round city homes. This has been made possible by the construction of modern pavements in place of the former dirt roads which existed in the rural districts of the State, and we find these citizens among the most prominent in the urging for the expenditure of large sums of money for wider roads, etc., which will permit them to get back and forth between their business in the city and their summer homes in the shortest possible time.

"Extremely heavy traffic on many roads is confined to holidays and week ends, and again we find it is the owners of city-owned vehicles for the most part who are leaving the city to spend the day at shore resorts, at country parks, or who are out for a ride and picnic party in the country. It surely would be little pleasure owning a motor vehicle in the city if one were confined to city streets in its operation."

Mr. Henderson advised against apportioning motor vehicle or gasoline tax fees to the use of cities, counties, or towns. He did not approve of using State funds for improving or constructing ordinary city streets, unless it were a definite and well-planned project designed to benefit the people of the entire State.

He concluded his argument as follows:

"Since the cities and towns set forth that they derive no income from these fees for use on the roads in said towns and cities, it is considered fair by some to argue that the tax collected by cities and towns on motor vehicles

as personal property is equal to or greater than the tax collected by the State as fees, and that this money collected by the towns and cities could properly be used for road purposes within the city or town limits."

## EQUITABLE DISTRIBUTION FOR HIGHWAY PURPOSES OF MOTOR VEHICLE LICENSE FEES AND GASOLINE TAXES

By Frederic E. Everett,* M. Am. Soc. C. E.

This paper was an exposition of the problem from the viewpoint of experience in the State of New Hampshire. The gasoline tax has spread in popularity since 1919 in a way Mr. Everett described as phenomenal. Professor F. G. Crawford, of Syracuse University, was quoted as saying,

"In only one or two cases has there been a decrease from one year to another and even where the rate has remained constant the gross amount has increased steadily. Increases in rates have caused corresponding increases in yield. This in part answers those critics who believed that increases in rates might result in evasions which would have a serious effect upon the total sum collected."

Further, Mr. Everett declared that,

"The gasoline tax is, without doubt, the most economical to collect, the cost in all cases being less than 1% with the exception of three States, that collect from the retailer, this method costing from four to five times more than it does to collect from the distributor. In New Hampshire there is no additional cost connected with the collection of the gasoline tax since it is done from the State Treasurer's Office in addition to the regular work.

"*Average Fees.*—Since the automobile has been accepted for general use, motor vehicle license fees have been collected by all States, the average ranging from $5 to approximately $30 per vehicle, the lower fees applying to those States having large registrations while the higher rates are found in most instances where there is large road mileage and low valuation.

"At the beginning of the present automobile age all funds for highway purposes were derived from taxation upon real and personal property, but with the growth of the automobile industry there has been an increased use of highways and a demand for a better type of road surface. Therefore, the automobile has been called upon to furnish a large part of the revenue to make these improvements possible."

Income derived from motor vehicle taxation and the gasoline tax should not be diverted to any other use than road purposes, according to Mr. Everett. While there are many different ideas as to the best way to finance various classes of highways, practically all agree that,

"The users of the highways should pay for their upkeep or for the maintenance of the primary system, which in most States includes trunk line highways carrying practically 75% of all the State's motor vehicle traffic.

"*New Hampshire Highways.*—Traffic studies in New Hampshire have shown that our trunk line highways (which total 11.3% of the State road mileage) carry 69.4% of all traffic. Our secondary system of State Aid roads, which total 13.7% of the State road mileage, carry 16.8% of the traffic; while the

* State Highway Commr., Concord, N. H.

town roads, which total 75% of the State road mileage carry but 13.8% of the total State traffic. On account of the heavy trunk-line traffic the construction must be of a high type or suffer a heavy maintenance cost. * * * When the original roads have to be rebuilt with a heavier type of construction to carry the increased traffic and the heavier loads forced upon them by modern trucks and buses, it seems only fair that the road user should pay the reconstruction cost unless it increases the valuation of the adjoining property. In many instances the reconstruction does bring about better conditions through the region adjacent to the improvement; thus, property valuation naturally increases and should rightly be assessed for its just portion of the improvement. In North Carolina improved highways have greatly increased the value of adjacent property.

*"Effect on Property Values.*—By contrast, New Hampshire has been building improved highways for twenty-five years and during that time has nearly completed the trunk-line system besides building about 900 miles of State Aid roads with improved surfaces equal to gravel or better. The increase of traffic in New Hampshire due in part to tourist traffic, which is more than 50% of the total, during the summer months, now demands that more than 60% of the trunk-line roads be rebuilt with wider and higher type road surfaces. This reconstruction program, with but few exceptions, will not increase property valuations to any great extent, nor will it create better school facilities or living conditions through the regions in which the improvements are made. Therefore, as the adjacent property owners will receive no additional benefit from the road improvement, why should they be subjected to an additional property tax?

"It is already proved that the motor vehicle owner will not object to his tax as long as he knows that it is being properly directed toward needed highway improvements. The reconstruction program in New Hampshire is being carried on by the aid of an increased gasoline tax which we believe is justified. Since more than 50% of our summer traffic is foreign, our visitors are helping us pay for the improvements which they enjoy as well as ourselves, since they are able to travel through our vacation-land on modern highways."

The simplicity of collecting gasoline taxes has caused a tendency to divert parts of the income for other uses than highways, but this trend has not spread far. New Hampshire practice in this respect is thus described by Mr. Everett:

"A certain amount of the gas tax money is re-apportioned to towns of low valuation and large road mileage for the maintenance of local roads which are not on the State system. This amount at present is close to $100 000 and is turned over to the towns to spend upon the maintenance of their local roads as they see fit. The State does have a town road supervisor, however, who confers with the local town road agents and gives expert advice when called upon to do so, but in most instances the money now paid over to the towns by the State is spent according to the wishes of the local town officers. Since the State highway systems are of primary importance to all road users, it would seem that the first thing for any State to do is to complete its main arteries by the State funds to be derived from the motor vehicles registration fees and gasoline tax or, if necessary, by a bond issue to be financed from these funds, as we are now doing in New Hampshire. Then when the main arteries have been built and if current revenues exceed the necessary maintenance costs the municipalities may be reimbursed by a certain portion of these funds. Such funds can then go for the betterment of local roads which are of little importance to the general State-wide system. Under no conditions should the revenues obtained from road users be expended for any other purposes than for bettering roads. Until our State-wide and interstate road systems are completed, it is important that all such funds should be expended under State control.

*"Six Methods of Distribution.*—Up to the present time there are at least six different methods used in distributing the fees obtained from taxes upon motor vehicles and gasoline. Twenty States return a portion to local communities, some refunding to counties, others to cities and towns. The method of distribution is in some instances based on road mileage, in others on valuation, while still others plan to divide the funds equally or in the direct proportion as they are received. However, as to which is the best method cannot be answered, for each State has its individual problems. The area of the State, its population, motor vehicle ownership, the number of counties, the road mileage of each county, together with analysis of the traffic—all enter into a solution of the problem. While in some instances the amounts proportioned to local communities have increased during the past few years, the greater amounts are in most cases being held by the States for centralized supervision of expenditures."

## DISCUSSION

### VARIOUS ASPECTS OF DISTRIBUTION

*Argument for the City.*—The city viewpoint was emphasized by J. E. Barlow, M. Am. Soc. C. E. His plea was for a more equitable distribution of the automobile tax income to the city. He recommended distributing the funds as between urban and rural highways, on the relative basis of ton-miles of use. He protested against charging property taxpayers with the burden of building and maintaining streets without any assistance from the users of the streets, that is, with assistance from motor vehicle taxes.

In Maine, the State collects not only the automobile fees, but also a 1-mill tax for highway purposes. Some of this is returned, but each year the City of Portland pays about $60 000 to the State Highway Fund. Mr. Barlow expressed the opinion that,

"This gasoline tax and the automobile taxes should be all collected by the State; it should be expended by the State on highways; and there should be no prejudice as against the cities and towns and the inter-rural thoroughfares."

This opinion was further supported by G. H. Norton, M. Am. Soc. C. E., who stated that in motoring to Boston from Buffalo, N. Y., it was in the cities that he was held up and not in the country. In his opinion,

"The expenditure on most of the State highways has produced a condition by which traffic may get over the intervening space much easier than it can get through the cities, because proper traffic facilities are not provided within the cities.

* * * * * * *

"The inhabitants of the city in many cases are the ones who are contributing very largely to the gasoline tax, and in addition to that they are paying a direct tax either through their State tax or local tax to provide these same highways through the city streets. To-day, the city user is paying entirely for the city street and is paying his full contribution for the amount he uses the State highway, through his gasoline tax; he is being doubly taxed."

In Buffalo, the city may now join with the county in improving certain streets and highways so as to provide major lines of traffic through the city. In Mr. Norton's opinion,

"The city should be allowed to collect an added tax if no other means are found whereby it can make its streets as suitable for traffic as many of our State highways."

In traveling to Boston it required a longer time for Mr. Norton to go the last 15 miles from Concord, Mass., than for any 45 miles across the State of New York in open country. Hence, his question,

"Why should not something be done to provide equal facility for getting in and out of a place rather than to make it so easy for them to get there and then be held up?"

*Taxing Commercial Traffic.*—How long will commercial traffic be carried on the highways without paying its proportion of the costs? George H. Kimball, M. Am. Soc. C. E., drew a parallel between highway costs and railway costs. The average mile of single-track railroad costs about $60 000 and a highway costs about the same; but a highway offers to commercial traffic a practically free right of way and roadway, items for which a railroad must ask a financial return. There are thirty-six or more routes of trucks that operate a freight and express business from South Station in Boston, to all parts of Massachusetts and beyond. "How long," asked Mr. Kimball, "will the taxpayer, on whom we all depend so much, stand the burden? * * * It is time that the State officials should consider that factor in the problem."

*Practice in Boston.*—As to the responsibility of the State in continuing and maintaining State routes through the cities, J. T. Whitney, Assoc. M. Am. Soc. C. E., agreed with other discussers. Until recently the State of Massachusetts spent very little money on thoroughfares in Boston, but that condition has been changed somewhat. In his own words,

"The State should construct certain thoroughfares which connect their own trunk lines through the cities or around the cities. Quite a large amount of money has been spent that way in the City of Boston within the last three years."

The Northern Artery was built by the Metropolitan District Commission with funds collected by a betterment tax. The Southern Artery, on the other hand, was constructed by the State Highway Department from funds raised by a bond issue. The cost was about $2 000 000, according to Mr. Whitney. It is located entirely within the Cities of Boston and Quincy for a distance of about three miles. To quote Mr. Whitney:

"Those two arteries, if necessary, should be connected by a thoroughfare to be built by the State either through the heart of Boston or around Boston on the south and west sides, and the City of Boston should not be compelled to pay any part of this improvement."

The people opposed the assessment for these two improvements and considerable litigation ensued until the Legislature finally passed an act removing the assessments and ordering refunds to be made.

*The Town Viewpoint.*—That the poor struggling town, as well as the city and State, have an interest in this subject, was emphasized by J. L. Murray, Assoc. M. Am. Soc. C. E.

In Rhode Island, he thought, the town was in a particularly unfavorable situation. Warwick, for example, is on the edge of Providence. In starting

out on picnics and outings, the traffic from the city often overflows to the streets of Warwick in an attempt to dodge heavier traffic, and for this wear on the streets the smaller towns should get some outside assistance. Mr. Murray suggested a distribution based on the town's valuation.

*Relation Between Gasoline Taxes and Pavement Costs.*—George B. Sowers, M. Am. Soc. C. E., reiterated his theory that the cost of building and maintaining a road pavement is very nearly in proportion to the gasoline consumption on a given unit of the road. He recommended that the gasoline tax should be distributed on a basis of the gasoline consumed per mile on the road.

It was not fair, he declared, to assess adjoining property owners as much as 98% of the cost of paving a thoroughfare. Specifically,

"If the main highway is through a residential street we would probably assess the property for 26 or 28 ft. of roadway of ordinary strength and the additional cost would be carried out by some general fund. * * * I do not believe it is fair to ask the taxpayer who pays upon the valuation of his property, to pay for the building of these highways throughout the State for the benefit of the motorists. A great many of the motorists are not heavy taxpayers. The truck companies and the pleasure car owners do not contribute much in the way of property taxes.

"The State of Ohio has recently put into effect a new Highway Law that will help to overcome some of the problems. The highway system extends through the cities, and the law permits the State to go into the cities and participate in the cost of improving these highways even to the full amount of the cost if necessary. It will help in the maintenance of the highways and also in the building of the bridges and viaducts through the city."

*A Separate Gasoline Tax.*—To apply a special gasoline tax for city revenue would not be feasible because motorists would simply drive out of town to make their purchases, according to Mr. Sowers. He was reminded that in Ohio, cities receive a refund of all registration fees and one-third of the gasoline tax. In Dayton, this equals as much as $325 000 each year. Cleveland also receives a considerable refund, declared Mr. Sowers, but it is all applied for maintenance of pavements and not for the construction of new pavements.

*Not a Luxury Tax.*—This was the view held by Mr. Edward S. Larned who emphasized the fact that, by taxing gasoline, the pleasure cars were taxed for the wear and tear far out of proportion to their effect. To quote directly,

"The gasoline tax has been said to be a fair measure of the use made of the highway. Well, it is, in a limited sense, but it is no measure whatsoever of the abuse of the highway, and the abuse of the highway can be traced directly to the heavy overloaded trucks that originate in the cities. We have express companies and freight companies that have established regular lines of business throughout the entire community and into the neighboring States, and that heavy trucking does far more damage to the State highways than all your pleasure travel. To be sure, they consume gasoline, but it is nowhere in ratio to the damage it does to our highways.

"I think the funds should be distributed by the State Highway Commission. I think the State Highway Commission is fully alive to the necessity of relieving congestion where the State highways pass through the city. In Massachusetts, the Highway Commission does render small towns State aid. It was the obligation on the part of the State because through traffic passing through a small community would utterly destroy the insufficient roads that

the communities were able to build in the first instance, and better roads had to be built. The State recognizes that fact and contributes to the expense. Trying to distribute the funds and fees by ton-miles will bring about impossible complications."

*Towns in Massachusetts.*—The State's obligation to the smaller towns was further outlined by L. O. Marden, Assoc. M. Am. Soc. C. E. He expressed the belief that,

"When the majority of the traffic on evenings and Sundays comes out of the cities and on to the country roads with the heavy burden upon these particular towns, that the towns should enjoy the use of the money from the motor vehicle tax."

Most of the cities in Massachusetts have pavements that can be used practically all year, declared Mr. Marden:

"In Worcester County, for example, we have 4 300 miles of road of which probably 1 200 miles, including the roads in the cities, have been improved. That leaves about 3 000 miles of unimproved roads, some of them in practically the same condition as during the Revolution. We have towns that are very poor, and it is difficult for them to raise even $2 000 to meet (we will say), the sum of $4 000 by the State and $4 000 by the county. The cities can generally raise the money for pavements through assessments or through betterment taxes."

*Distribution of Funds to Minor Political Sub-Divisions.*—The trend of the discussion led Chairman H. K. Bishop to call attention to questions raised by C. E. Myers, M. Am. Soc. C. E.,* who emphasized the fact that,

"If the certain proportion of the gasoline tax were to be allocated to the cities it would have a tendency to reduce other taxes and, instead of devoting it to road purposes, the city would simply cut off from other sources certain taxes to equalize the income. There would then be no more money spent for road purposes within the city, and the source of obtaining the taxes would be better."

## TRAFFIC CONTROL IN MASSACHUSETTS

By Maxwell Halsey,† Esq.

*State Efforts.*—Extensive studies of highway traffic on the part of State authorities, as described by Mr. Halsey, have resulted in questionnaires, regulations, codes, and traffic surveys. These studies are now bearing fruit. He explained the origin and development of this work, together with many details of its execution.

Active State interest in traffic was authorized by a law which became effective in June, 1928. Its purpose was threefold, according to Mr. Halsey:

"First, the elimination of confusion to the motoring public through the accomplishment of a basic conformity in the signs, signals, and traffic control devices used by the Commonwealth and by cities and towns.

"Second, the maximum safety and facility of through traffic movements upon State highway routes lying within the corporate limits of cities and

* *Proceedings*, Am. Soc. C. E., August, 1929, Papers and Discussions, p. 1396.

† Traffic Engr., State Dept. of Public Works, Boston, Mass.

towns: (*a*) By preventing unnecessary obstructions of such routes by improperly placed and operated control devices; and (*b*) by protecting such through routes from dangerous interference from cross-traffic by application, where necessary, of the 'through traffic stop' rule.

"Third, the creation of a responsible traffic engineering agency in the Department of Public Works, capable of co-operating with local officials in developing safe and uniform methods of traffic control and administration."

*Procedure.*—Thus, it was hoped to afford definite and rational leadership to all communities in their efforts to improve and standardize traffic regulation. This was effected partly through the Albert Russell Erskine Bureau for Traffic Research, at Harvard University.

As Mr. Halsey related, information was broadcast throughout the State to each Chief of Police, explaining the scope and purpose of the efforts. In turn, questionnaires, 300 or more in number, were sent out on which reforms or unified practice could be based. Besides approval of proposed installations, the State Department has advised fifty cities and towns, on request.

Another result of these activities was the formulation of "Bulletins" covering mooted points. That on "Through Traffic Streets" attempted to alleviate a pressing problem. It showed, for example, the purpose of controls, proper conditions for success, and the desirable types of signs.

*Other Studies.*—Among the various activities of this State Department is a study of accident records, to show by analysis just what factors contribute to safety. It is hoped that this will influence the types of construction, lighting, and the erection of danger signs.

Still another effort has been directed toward a State-wide traffic count. This, Mr. Halsey explained, was to show "general flow conditions" and give a "traffic pattern of the State." At the same time, other data referring to speed and obstruction showed the particular roads which needed control and just what were the critical points. A sample form used for such a traffic count is shown herewith.

*Adoption of Through Trafficways.*—One of the important functions of the State Department, according to Mr. Halsey, was to designate, where necessary, through ways, and to give them the required protection to expedite traffic.

In this same connection the Department is empowered to establish traffic control signals. For this purpose, the accident data and the flow studies previously made have been found valuable. In general, as Mr. Halsey explained,

"The Division has followed the policy of dealing only with authorized public officials, permitting them to take the initiative in all matters. Cordial relationships have been made with all sign and signal companies who are interested in this State movement toward uniformity. Special effort has been made to give immediate attention to all current requests for permits.

* * * * * * *

"In all cases involving other Divisions of the Department or other Departments, they are consulted and their advice solicited. Complete records of all transactions, procedure, and plans are being kept available for immediate reference."

*Control Signals.*—One of the noteworthy achievements of this work has been the formulation of a standard code covering traffic control signals. On

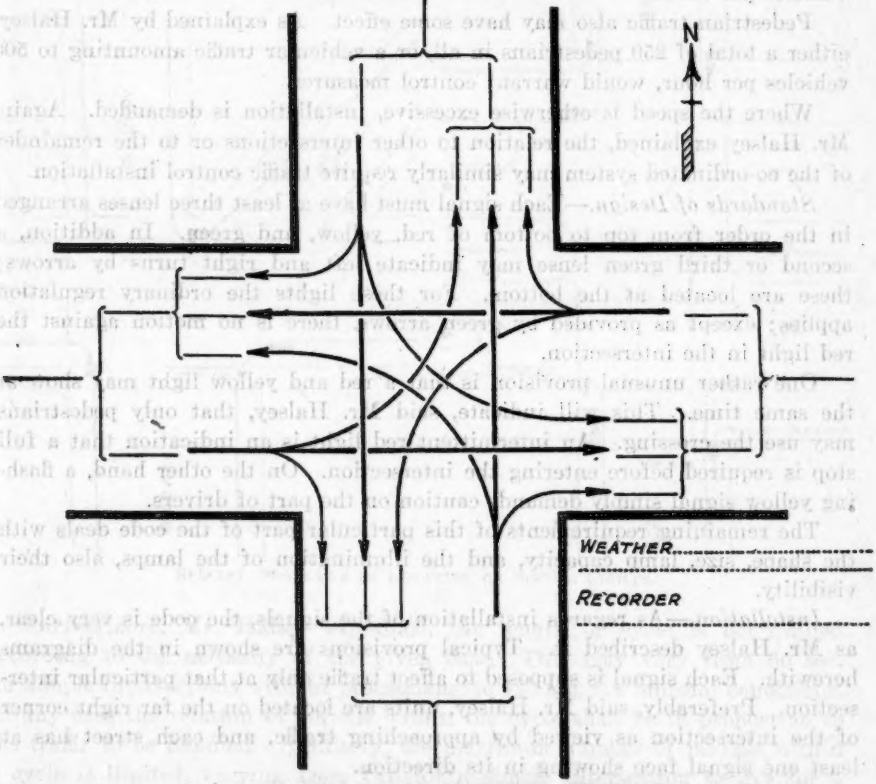
this matter Mr. Halsey went into considerable detail explaining the "stop-and-go" signals according to the State code, examples of different methods of installation, and the procedure followed by cities and towns to install the proper signals.

## TRAFFIC COUNT FORMS

COMMONWEALTH OF MASSACHUSETTS
DEPARTMENT OF PUBLIC WORKS
Room 413 State House, Boston.

CITY OR TOWN.....
LOCATION.....
.....
DATE OF COUNT.....DAY.....

INTERSECTION COUNT OF VEHICULAR TRAFFIC
SUMMARY SHEET




DATA FOR SIGNAL TIMING

| ENTERING TRAFFIC (NAME OF STREET) | TOTAL FLOW (Number of hours.....) | | PEAK HOUR FLOW (Time.....) | | CHECK | |
|---|---|---|---|---|---|---|
| | NUMBER | PER CENT | NUMBER | PER CENT | INBOUND | OUTBOUND |
| Important street | | | | | | |
| Side street | | | | | | |
| TOTAL | | | | | | |

EXAMPLE OF REPORTS FOR TRAFFIC COUNTS.

*Standards of Utility.*—Referring directly to the State code, as given in *Traffic Bulletin No. 2,* Mr. Halsey quoted the regulations regarding "Standards of Utilities." The first requirement is that "signals may be installed or operated only where and when conditions warrant their use". There is a certain reasonableness, he explained, which gives warrant for methods of control. It is always possible that too much control will be attempted, and this will defeat its own purpose.

Furthermore, "signals may be installed and operated only where, and during such hours" as, certain conditions exist. For example, the total volume of traffic at the intersection must be equal to 500 vehicles per hour; or that part of the total traffic entering from a minor street must be equal to 125 vehicles per hour.

Pedestrian traffic also may have some effect. As explained by Mr. Halsey, either a total of 250 pedestrians in all, or a vehicular traffic amounting to 500 vehicles per hour, would warrant control measures.

Where the speed is otherwise excessive, installation is demanded. Again, Mr. Halsey explained, the relation to other intersections or to the remainder of the co-ordinated system may similarly require traffic control installation.

*Standards of Design.*—Each signal must have at least three lenses arranged in the order from top to bottom of red, yellow, and green. In addition, a second or third green lense may indicate left and right turns by arrows; these are located at the bottom. For these lights the ordinary regulation applies; except as provided by green arrows, there is no motion against the red light in the intersection.

One rather unusual provision is that a red and yellow light may show at the same time. This will indicate, said Mr. Halsey, that only pedestrians may use the crossing. An intermittent red light is an indication that a full stop is required before entering the intersection. On the other hand, a flashing yellow signal simply demands caution on the part of drivers.

The remaining requirements of this particular part of the code deals with the shape, size, lamp capacity, and the illumination of the lamps, also their visibility.

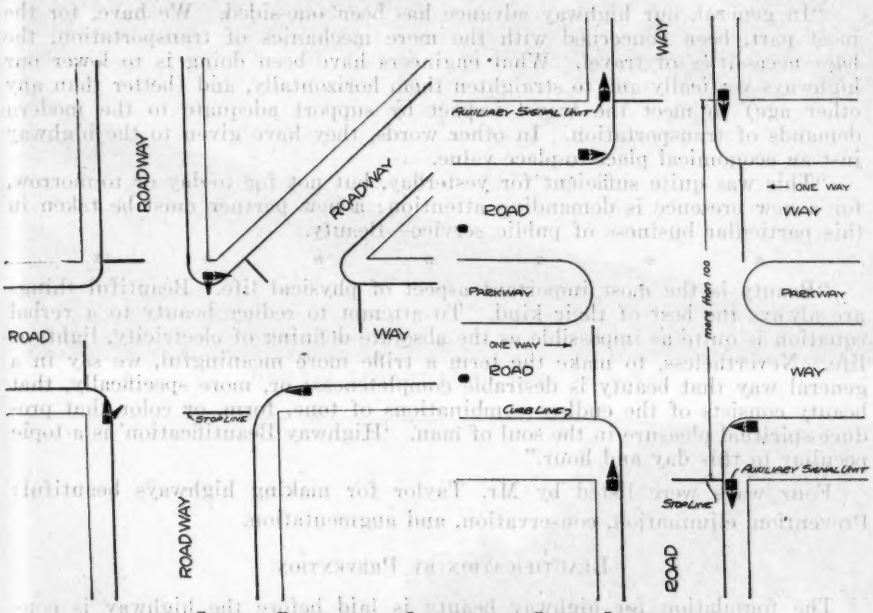
*Installation.*—As regards installation of the signals, the code is very clear, as Mr. Halsey described it. Typical provisions are shown in the diagrams herewith. Each signal is supposed to affect traffic only at that particular intersection. Preferably, said Mr. Halsey, units are located on the far right corner of the intersection as viewed by approaching traffic, and each street has at least one signal face showing in its direction.

As indicated on the diagram, each signal is placed as close to the traffic it will control as is practicable. In height it shall be such that the bottom is between 8 and 16 ft. above the roadway, but in any case less than one-fifth the distance to the stop line of the traffic it controls, with 8 ft. as the minimum. As to line, the unit shall be within 1 ft. 6 in. of the outer edge of the curb.

If these provisions bring the standard signal location more than 100 ft. from the stop line, an auxiliary signal at the nearer property line usually shall be added. Furthermore, a line of vision of the signal for at least 150 ft.

is required before the driver reaches the stop line, and each signal is shielded to insure that it shows only in the direction for which it is to control the traffic.

*Operation.*—One requirement, according to Mr. Halsey, is that signals must operate at all times. This may either be by absolute control (stop and go), or as warning devices, showing intermittently and providing for traffic under caution. In case the operation is discontinued for any time, the signal must be taken down entirely, or the faces must be hooded so that lenses will not be visible. Between the hours of 11:00 P. M. and 7:00 A. M., they shall be turned to flashing operation instead of direct control; this, however, does not apply to certain business intersections.




SPECIAL PROBLEMS IN LOCATION OF SIGNAL LIGHTS.

Furthermore, Mr. Halsey explained, the length of cycle is determined according to the necessity of the given case. This may vary from 60 sec. for simple intersections without pedestrians, to 120 sec. for unusual conditions. In any case the relation of periods within the cycle shall be in proportion to the traffic to be handled. Similarly, the maximum number of periods within a cycle is limited, varying from two for a simple intersection to five for a special condition. A change period (yellow light) must be used following a green period or pedestrian period, and may be used following a red period. It shall be from 3 to 5 sec. in duration.

Co-ordination within a system of traffic signals is permitted according to the necessity of each case. Where the system is progressive, the speed of progression must agree with that permitted by law.

Following his address, Mr. Halsey submitted to a rapid fire of questions covering many of the details mentioned. A marked interest was shown in

this movement, and the comment exhibited general approval of the program and of the details of its operation.

## HIGHWAY BEAUTIFICATION

By James H. Taylor,* Esq.

While the present generation may not be producing better architecture, sculpture, paintings, music, or books, there can be no doubt that it is producing better roads, according to Mr. Taylor. Nevertheless,

"In general, our highway advance has been one-sided. We have, for the most part, been concerned with the mere mechanics of transportation, the bare necessities of travel. What engineers have been doing is to lower our highways vertically and to straighten them horizontally, and (better than any other age) to meet the travel impact by support adequate to the modern demands of transportation. In other words, they have given to the highway just an economical place-to-place value.

"This was quite sufficient for yesterday, but not for to-day or to-morrow, for a new presence is demanding attention; a new partner must be taken in this particular business of public service—Beauty.

* * * * * * *

"Beauty is the most important aspect of physical life. Beautiful things are always the best of their kind. To attempt to reduce beauty to a verbal equation is quite as impossible as the absolute defining of electricity, light, or life. Nevertheless, to make the term a trifle more meaningful, we say in a general way that beauty is desirable completeness; or, more specifically, that beauty consists of the endless combinations of tone, form, or color that produce spiritual pleasure in the soul of man. 'Highway Beautification' is a topic peculiar to this day and hour."

Four ways were listed by Mr. Taylor for making highways beautiful: Prevention, elimination, conservation, and augmentation.

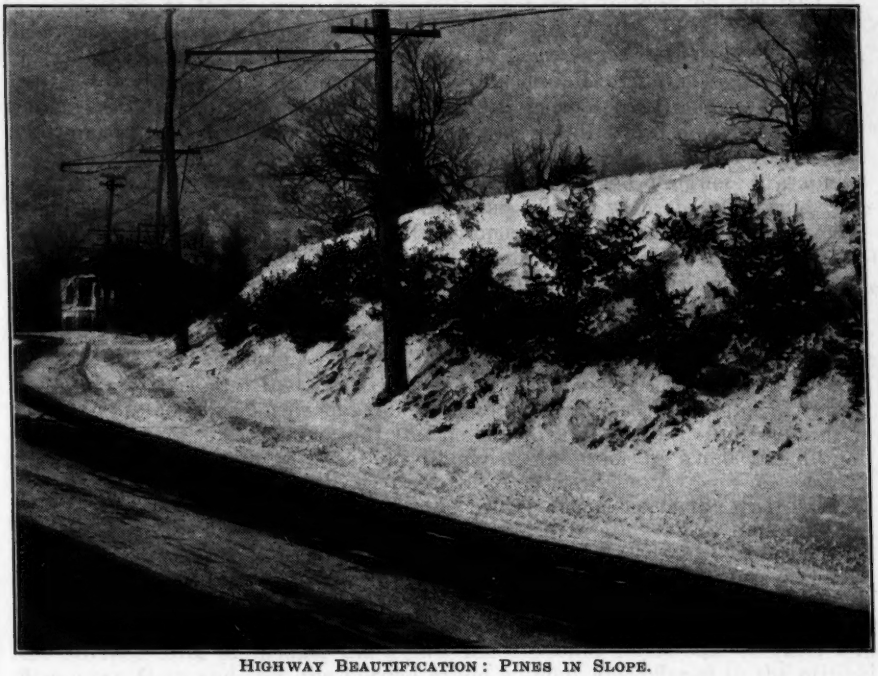
### Beautification by Prevention

The foundation for highway beauty is laid before the highway is constructed—in the attitudes of the designer and the locating engineers. Their function is to prevent mistakes that would have to be corrected after construction. Mr. Taylor argued that,

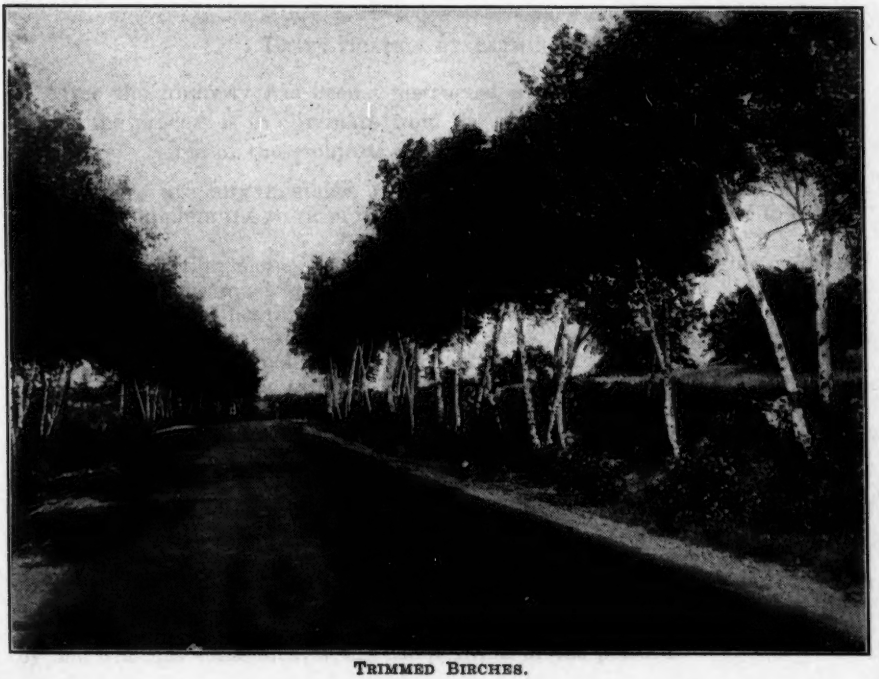
"A field party chief whose big aim is a traverse check, is like the proverbial 'bull in the china shop'. His men, with never a thought, chop great scars in the tree trunks to locate (very accurately no doubt) their tie points. What does the stolid assistant care for beauty? He could have thrown his line a few feet from that row of splendid maples, but it never occurred to him. If you are going to keep men like that on the payroll, your particular 'Highway Beautification' will progress slowly. The highway should have a stream line; there should be no breaks. It should flow smoothly.

"It is not an uncommon thing to find a very competent engineer whose vision is bounded by his drawing board. One of the finest 'first aids' toward the improved highway would be to give the inside assistants an outside trip, and frequently. Let the stenographer see what she is writing about, and the draftsman what he is planning. Highway beautification can progress only

* Highway Landscape Superv., Palmer, Mass.



HIGHWAY BEAUTIFICATION: PINES IN SLOPE.



TRIMMED BIRCHES.

in proportion as it is served by generally awakened interest on the part of those primarily related to it.

"In the pursuit of highway beauty it is necessary for the designer constantly to visualize the ultimate effect of his design. In surface alignment, the curves must be adequate and the tangents part of a harmony. In grade, there must be no unexplainable 'lump' that the eye of travel detects instantly. We must constantly keep in mind the total effect of the completed road.

"*Bridge Designers With an Eye for Beauty.*—Bridge designers are apt to be men who are merely mathematicians; very clever with the slide-rule, but totally ignorant of the rule of beautiful proportions. Bridges should be something more than a structure of designed interior strength; they should be designed for external beauty as well. No bridge should be designed without competent architectural assistance. The structures of the older nations should guide us. The lines of these permanent works are to delight or disgust the eye for centuries to come.

"We must see also that construction serves our end. Tree trunks must be properly protected. Shovels and rollers should be equipped with flame arresters. That shell hole of construction, the borrow-pit, should be studied, and, if possible, entered from the rear or otherwise, in order to avoid the all too common scarring of the highway picture.

"Many mistakes are made when the bridge fence meets the guard fence. On tangent, the fill should be of sufficient width to allow the guard fence to follow the bridge fence line. On a curve, the guard fence must be concentric with the surface lines. In the case of a vertical treatment of fence jointing, where the bridge grade is higher than the normal surface grade, the grade approaching to the bridge by a gradual reverse curve merging smoothly into the surface level of bridge, the guard fence should follow the surface, and flow away from and into the normal grade without any offence to the critical observer."

## Beautification by Elimination

After the highway has been constructed and put in operation, the next step in the process is to eliminate from the scene all objects that tend to mar its beauty. Some of these objects were mentioned by Mr. Taylor:

"If there are any mistakes through poor design, poor workmanship or failure to complete the work properly, maintenance must do its best to correct the conditions.

"If construction money has failed to provide for tree care after construction, maintenance must 'get busy'. Stumps left a foot or two high must be removed. If beautification is to be served faithfully we must get every dead thing off the highway—dead wood, dead tracks, dead watering troughs, dead signs, dead wire, dead poles, dead contractor's junk.

"The 'bump' at the railroad crossing must be smoothed out and, after the settlement is complete, the surface drop on either side of its bridge walls should be corrected. Then, there is the grass and weed mowing that we are apt to forget; also the specially undesirable weeds, burdock, yellow dock, mullen, and the like. If allowed to mature, they are not good to look upon and spread their seed. Trees and brush on the inside of curves must be cut away for safety and kept out. Safety first—beauty second.

"There is no reason why beautiful brooks and rivers should be allowed to run behind screens of trees when a little axe and saw work will give us beautiful pictures. This is one way of the finest types of highway beautification by elimination. The cutting out of such views, however, must be done by one who has some idea of the composition of the picture.

"In the past, the chief destroyers of roadside comeliness have been the wire companies. Trees mean nothing to them but nuisances; * * * but that sort of thing is all over. * * * No wire company can lawfully touch a tree without permission and then under the eye of the State Tree Inspector. This does not mean that the tree does not suffer. Wires and trees cannot live together without fighting. The tree always loses. The public loves trees, but it loves service more. The result is a compromise which is not ideal, but the best we can do at present. The model highway is one wide enough to give wire, water, gas companies, etc., a space where they can function, behind a planting of trees.

"Attention is called to another improvement by elimination—the replacing of the 50-ft., 8-armed, 10-pin poles of a few years ago by 25-ft. poles carrying the wires in a cable and running half concealed among the roadside trees.

"As a result of a definite program, trees in Massachusetts are receiving considerable care. There are very few dead trees on our roads. A roadside tree has a tough fight for life; the wires above, the pipes below, and the automobiles midway, make every day an increasing hazard. We are giving injured trees as much attention as possible—cabling the splits, filling the cavities, trimming and painting wounds, straightening, pruning, and shaping them, giving them more of a chance. Trees are man's best servants and deserve man's best attention."

### Highway Beautification by Conservation

Clearing the roadside requires a technique based on experience so that trees and shrubs that would add to the natural beauty may be conserved. In the words of Mr. Taylor,

"The roadside should be studied as to selective cutting out, the hardwoods and evergreens being given the preference. * * * When a desirable tree is noted the surrounding growth should be cut away and the sapling correctly trimmed and encouraged into a proper growth.

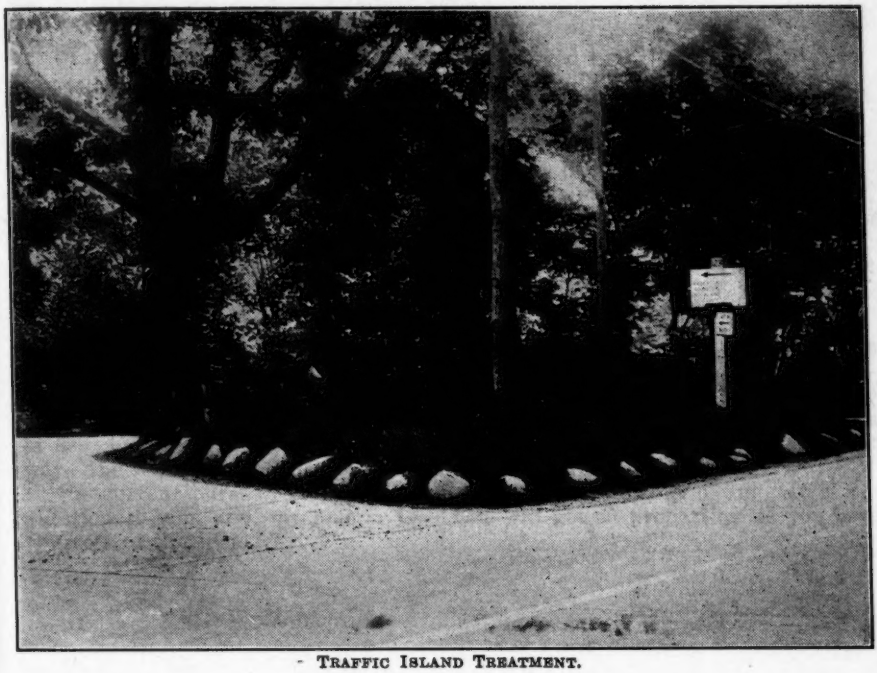
"It is, of course, apparent that highway beautification in any of its phases is possible only as a result from some degree of education. This is especially necessary in this particular stage. The workman must be handpicked and must know the treasures of his roadsides. Only men with a real interest for and in the work are worth while. Given just a little to build on, such an interest is easily established. I find that some of our men, who a few years ago knew very little about roadside growth, are to-day reading into the subject, buying expensive books, using the cultural terms; in short, acquiring a roadside technique. And it must be so, for how can a man develop a roadside, unless he is familiar with its factors?"

### Beautification by Augmentation

In general, said Mr. Taylor, the best practice is to avoid artificiality and keep the roadside as natural as possible:

"Introduced trees, shrubs, vines and plants will be used with increasing intelligence as the highway beautification idea moves on.

"The monotonous planting of mile after mile of straight rows of evenly spaced trees will not serve the coming generation. More and more we will observe Nature's inimitable way. There can be no better teacher. The scarlet of the woodbine on the gray rock, the glowing lily against the lightened wall, are very small but wonderful touches."



TRAFFIC ISLAND TREATMENT.

THE GLORY OF COMMON THINGS.

TRAFFIC ISLAND TREATMENT.



THE GIANT OR COMMON THISTLE.

## An Ultimate Ideal

In concluding his remarks, Mr. Taylor observed that,

"Hitherto, highways have consisted in three parts: Line, grade, and surface. The age requires an additional factor—beauty.

* * * * * * *

"The pioneer stage of highway construction is history. The main problems of transportation have been solved. We have met the impact and held it. There will be no new surface materials. The hard work is over. We have reached a contemplative period of highway history, when we can, with the more leisure, recapitulate and by virtue of what we have done, conclude what may be done better.

* * * * * * *

"Your splendid highways in their present frames, beautiful as they are without a thought of æsthetic improvement, are amply foundationed to continue as able ministrants to the mere physicality of living. They will carry just as many loads of ashes, just as many bales of rags, just as many pounds of fish; but to my mind, I have suggested the step beyond—the more excellent way—the refinement of method—the charm of completeness.

"A current phrase runs, 'What are you going to get out of it?' It is my question to you, and, my answer is—'You will get in proportion as you give yourself to the ideal.'"

# SURVEYING AND MAPPING DIVISION

OCTOBER 10, 1929—2:15 P. M. TO 5:00 P. M.

## SURVEYS FOR SWIFT RIVER RESERVOIR OF THE BOSTON METROPOLITAN DISTRICT WATER SUPPLY

By N. Le Roy Hammond,* Esq.

The proposed Swift River Reservoir to be constructed for the Metropolitan Water District of Boston will collect water from an area of about 186 sq. miles. Of this, 88 sq. miles will be required for reservoir purposes, including practically the entire Towns of Enfield, Greenwich, and Prescott, as well as parts of at least seven other towns in Central Massachusetts.

In describing the region, Mr. Hammond stated that the population in the Swift River Valley is only about 20 persons per sq. mile and that the land is covered mostly by woodland, varying from scrub growth to mature timber. Beginning late in 1926, topographic surveys were made within the basin of the proposed reservoir, extending at least 10 ft. in elevation above any probable flow line. At the same time, property line surveys and surveys to determine real estate holdings, etc., were being made.

*Triangulation.*—The general terrain of the valley was found to be ideal for a good system of quadrilateral network, except that considerable difficulty was encountered in placing stations so as to be readily visible along the axis of the valley.

The Massachusetts Harbor and Land Commission had published in 1912, the geographical position of a number of triangulation stations and this information was found valuable. Mr. Hammond declared his belief that,

"Few engineers appreciate the quantity of accurate information and data available within relatively small areas for a check and tie-up of land surveys of all descriptions. Throughout the Commonwealth of Massachusetts are many triangulation points of known geographical location, also all town corners with careful description supplemented by actual survey sketches. A more general use of the data at hand would do much to correlate the many surveys which are constantly being made."

*Field Methods.*—In referring to Swift River, mention was made of the East, Middle, and West Branches, running generally southward, so that,

"The general network consisted of two chains of quadrilaterals covering two of the major valleys extending generally parallel in a northerly and southerly direction. At their upper and lower ends, the quadrilaterals were brought together in order that a proper check might be available for the entire system. In a few cases, polygons with an interior station were included in

* Div. Engr., Metropolitan Dist. Water Supply Comm., Enfield, Mass.

the system as well as a few simple triangles to make control accessible to certain districts. Common tripod signals, about 25 ft. high, were permanently erected on each of the stations of the system. The signals were so constructed as to permit setting the transit directly under the signal mast. Angles were measured by an engineer's transit graduated to minutes. Six repetitions of each angle were made and the error of closure of the figures of the network was generally kept well under 8 sec. The triangulation network was laid out with the requirement that no angle be less than 27° and with triangles as nearly equilateral as possible.

"A base line about 1 mile in length was precisely chained by the slope method of chaining, using a standardized Chicago tape with spring balance. From the base line the system was developed to the major net of the triangulation. By utilizing the old existing triangulation points in the network of the system, with the published locations of these stations, it was possible to employ several checks to the new system. From the main network of triangulation, called the primary system, secondary triangulation stations were located as the needs of detailed survey developed."

*Co-Ordinates.*—By establishing axes of reference far enough outside the areas surveyed, a system of co-ordinates was determined such that all the values were positive. For this purpose, the plane of reference was assumed tangent to the earth at an arbitrary point near the center of the region. Specifically,

"All vertical co-ordinates are parallel to a tangent to the meridian, 72° 18′ 00″, west of Greenwich at its intersection with north latitude, 44° 22′ 30″ and all horizontal co-ordinates are parallel with a tangent to the parallel of north latitude, 72° 18′ 00″."

The co-ordinates of all established triangulation points were computed, the direct azimuths calculated between points, and the information was listed for the convenience of field parties. The distances between triangulation stations were short enough so that,

"Traverses on true azimuth were run into the area to be surveyed from either the primary or secondary triangulation legs. Practically all traverses have been run by use of the stadia, calculated, and closed back to the triangulation system or other checked traverses."

*Topography.*—Bench-marks were established by a line of precise levels transferred from the Wachusett Reservoir. Levels were taken from these bench-marks by vertical angles and topography was located entirely by the stadia method. Ultimately, said Mr. Hammond, about 28 000 acres of land will be thus surveyed and enough detail is recorded to permit plotting contours at 5-ft. intervals on a scale of 1 in. = 200 ft. While the topographic survey included information that would be useful in the real estate survey, the two were conducted separately.

*Real Estate.*—The Commission is empowered to acquire land by right of eminent domain, but to date (October, 1929) that has not been found necessary. Ultimately a general "taking line" may be established accurately and all the area bounded by the line will then be acquired. During the years in which the Enabling Act was being prepared and discussed, most of the property owners affected, were made cognizant of the need for their holdings and many of them subsequently offered the land for sale. When a parcel is so offered, according to Mr. Hammond:

"Its location is approximately determined by the field office in order that need for its purchase may be determined. The property is then appraised by agents of the Commission and steps taken for its purchase. Prior to the actual transfer of ownership, the title of the property is examined by the Legal Department and if metes and bounds or acreage appear obscure or inaccurate, surveys are requested of the engineering office. * * * In the actual surveys, a rigid requirement is enforced in that at least two independent locations be made of each property corner where corners are located as side shots from a main traverse and are not included in the traverse calculations. Such a method of location serves as a check when plotting the data of location.

"Plots are made to a scale of 1 in. = 200 ft. All traverses are checked back to themselves or other checked traverses, lines being run on a true azimuth derived in the beginning from the triangulation net. As often happens in undeveloped country such as is here encountered, it has been found that exact property lines are more or less obscure even to the knowledge of the owner and it becomes necessary to examine carefully deed and title of abutting owners in order that all property in the vicinity of the surveyed parcel might be satisfied."

The Legal Department has examined titles on properties that might be needed in the future as well as those required at once. Photostats of title abstracts are supplied to the engineering force, who are thus kept continually informed and are in a position to push the survey work in a systematic manner.

One practice of especial interest is described by Mr. Hammond, as follows:

"Together with the preparation of maps for actual acquisition of real estate by deed, an analysis of the property is made by the engineering office. That is, a review of all lines is written, with its relation to deed held by owner as well as to deed held by abutting owner. Assistants have acquired a much broader outlook on real estate survey requirements by being obliged to actually write out their analyses for review by the Legal Department. A suggested description for possible deed conveyance also accompanies a return of the map of the property. This requirement has the tendency to make certain that necessary bounds, line of occupation, distances, etc., appear properly on the map before it is passed to the Legal Department. The requirement of map, analysis of deeds, and suggested description, appeared at first to be somewhat burdensome to an engineering force, but as such analyses would be required mentally in the process of survey, the actual writing of the story soon became a comparatively simple matter and has been extremely valuable not only in the training of personnel to proper real estate survey, but also in the placing on record of the reason for each bound, line of occupation, or map line embodied in the final map."

*Aerial Surveys.*—The work of the aerial surveyor, prior to any ground work, was given an appreciative mention by Mr. Hammond. Contact prints were furnished of 100 sq. miles, photographed at an elevation of about 12 000 ft. above sea level. The resulting scale was approximately 1 100 ft. to the inch on 7 by 9-in. sheets. The contract provided also for prints enlarged to a scale of 1 in. = 400 ft. It was found, however, that even on single contact prints the elevation varied as much as 500 ft. in some cases. The resultant camera distortion made the enlargements of little value.

Nevertheless, Mr. Hammond made it plain that,

"By considerable study for the proper placing of ground control, ratios could have been deduced so that a reasonably accurate mosaic could have been developed. For the purpose of our work it appeared that the contact prints in

themselves were sufficiently clear and capable of being pantographed as the ground survey developed a control for the needs of the work. The contact prints have been in more or less constant use since their receipt and have been extremely valuable in many ways.

*"Value of Contact Prints.*—An inspection of aerial contact prints indicates, by lines of occupation or by variation in timber growth, the possibility of proper location in the field. In cases of three of the towns affected by the Swift River Reservoir such lines were pantographed to a skeleton map of the town. With these data on the map a perambulation of the town was made in company with its assessors to determine approximate ownership of lands within the town. Lines which had no significance as property lines were crossed out in the field and a somewhat crude map of property ownership within the town was developed. Such maps have been of considerable advantage to title examiners in the tracing of ownership titles. The maps were executed simply as an aid to an accurate ground survey of the area."

*Cemeteries.*—On all projects of this kind, the transfer of cemetery sites always presents many interesting problems. Mr. Hammond's account of one such case is given in full, herewith:

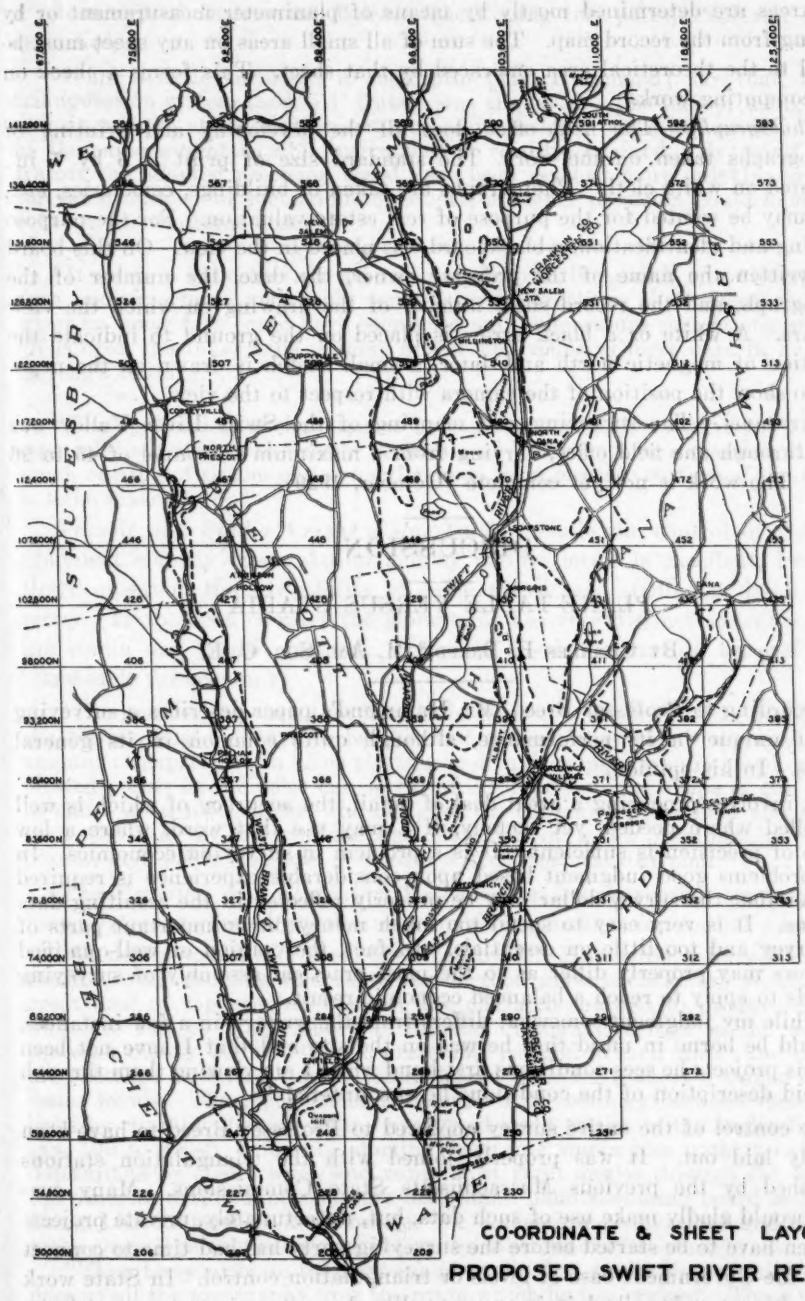
"Within the area required for reservoir purposes are located several cemeteries which will require removal outside of the reservoir area. Each cemetery has been surveyed, including a location of all monuments and markers together with the exact location of the indication of all burials. Copies have been made of inscriptions on all stones and so far as possible data gathered as to actual lot ownership. A census of burials in the cemeteries to be removed indicates the existence of more than 5 000 interments.

"A card index system has been developed covering the data of each interment, together with information concerning lot ownership. In many cases the latter data are more or less obscure due to the absence of old town records, but as actual removals are made it is probable that public interest will make known much of the missing history of actual interments and of interested parties."

*Mapping.*—The field notes for all classes of surveys are plotted together on standard sheets (24 by 36 in.) of white paper, mounted on cloth. These sheets are designed to match, edge for edge, and there is no overlapping. Standard titles and the graphical scale are printed on the sheets before they are delivered to the field office. Each area plotted is given a number, corresponding to the consecutive map number. The margins are made parallel to co-ordinate lines and detailed data are plotted by means of a paper protractor with an arm, or by drafting machine.

Two tracings are made from each drawing: One of the topographic features and the other of real estate holdings. The advantages are best described in Mr. Hammond's own words:

"With the early assignment of sheet numbers to areas included within definite co-ordinate limits, it became easily possible to control for reference, the location of any data taken. Triangulation stations were given numbers which corresponded to the number of the record sheet on which they would occur and all data taken in the field have been correlated to their proper sheet number. Loose-leaf notebooks have been in use and the data taken are grouped together to form books having a number the same as that of the record sheet on which the data would occur. This system has simplified the matter of keeping in order, the information secured. If a field party is assigned to a definite task, it can, by locating the sheet number of the area in which it is interested, immediately secure all available data which have been taken previously in that particular area and can reduce its own work to a minimum."


CO-ORDINATE & SHEET LAYOUT
PROPOSED SWIFT RIVER RESERVOIR
1929

Areas are determined mostly by means of planimeter measurement or by scaling from the record map. The sum of all small areas on any sheet must be equal to the theoretical area embraced by that sheet. This forms a check on the computing work.

*Photography.*—The field office does all the developing and printing of photographs taken on the work. The standard size of print is 5 by 7 in., mounted on white cloth. Photographs are taken of buildings, cemeteries, etc., that may be wanted for the purpose of real estate valuation. For the purpose of filing and identification, a blackboard was placed in the view. On this board are written the name of the property owner, the date, the number of the photograph, and the record sheet number of the drawing on which the view appears. A white or a black arrow is placed on the ground to indicate the direction of magnetic north and, later, a small sketch is drawn on the negative to show the position of the camera with respect to the view.

*Personnel.*—The surveying and mapping of the Swift River Valley was done through one field office, varying from a maximum personnel of 45 to 26 men. The work is not yet complete (October, 1929).

## DISCUSSION

### PLANE-TABLE VERSUS STADIA

By Charles B. Breed,* M. Am. Soc. C. E.

According to Professor Breed, Mr. Hammond's paper describes a surveying project unique in its performance, although quite common in its general aspects. In his opinion,

"It involves procuring a great deal of detail, the accuracy of which is well controlled where needed, yet sketchy, if I may use that word, where a low degree of precision is sufficient. It is a problem in surveying economics. In such problems good judgment based upon considerable experience is required in order that the survey-dollar may be properly reflected in the resulting data or plans. It is very easy to spend too much money on unimportant parts of the survey and too little on essentials. In fact, the opinion of well-qualified surveyors may properly differ as to the most practical assembly of surveying methods to apply to reach a balanced economic result.

"While my judgment somewhat differs from the writer's in a few instances, it should be borne in mind that he was on the site and that I have not been upon his project; he sees conditions first-hand while I am viewing them through his lucid description of the conditions he encountered."

The control of the entire survey appeared to Professor Breed to have been logically laid out. It was properly joined with the triangulation stations established by the previous Massachusetts State Commissions. Many surveyors would gladly make use of such data, but, unfortunately, private projects too often have to be started before the surveying party has had time to connect it with the Government base of levels or triangulation control. In State work of the character described it is more readily done than in private projects, because more time is often permitted for preliminaries.

* Prof. of Railway and Highway Eng., Mass. Inst. Tech., Boston, Mass.

*Accuracy of Instrument Work.*—Questioning one point, Professor Breed remarked,

"I am a little surprised to hear him [Mr. Hammond] say that, in the triangulation system used, a 1′ transit was employed. The idea of using a 10″ transit may be abandoned on account of its weight, but a 30″ or 20″ transit of the same weight as a 1′ instrument is readily obtained. By using a 20″ transit the adopted precision could have been reached more quickly; or, with the same number of repetitions of angles, a higher precision could have been reached for the same cost.

* * * * * * *

"It seems a little unusual to measure the base lines with an ordinary steel tape and determine and apply temperature corrections throughout the work when an invar tape could have been used and temperature corrections omitted. Of course, the invar tape is more brittle and requires more care in handling in rough country.

* * * * * * *

"The application of the plane co-ordinate system of location of all points is especially adapted to the ready replacing of lost points in a district where so much cutting of timber and general change in immediate surface of the terrain is to be made."

*Transit and Stadia Versus Plane-Table.*—While the control of the topographical map by triangulation and by precise levels is excellent, Professor Breed questions the advisability of using transit and stadia for detail topography. In the open country the plane-table has very distinct advantages over the stadia and transit methods. Even in wooded countries, he prefers the plane-table for economy:

"The number of points that have to be located is much smaller; the shape of the terrain ought always to be sketched while it lies before the eyes, as no amount of experience in office plotting and in drawing contours based on many plotted points in the office can so faithfully represent the shape of the ground as can a good topographer who sketches the contours in the field and governs their accuracy by the position and elevation of a few intelligently selected points; and, lastly, the cost is less for a given accuracy of plan."

*The Cruising and Sketching Method.*—As a constructive comment Professor Breed suggested an alternate surveying method for special conditions:

"In dense woods, like those described by Mr. Hammond, I have found that the following cruising and sketching method gives satisfactory results for a great deal of topographic work of the character he had to produce. Stadia lines are run by plane-table and alidade through clearings in the woods at intervals of, say, 500 to 5 000 ft., preferably parallel to each other; and contours are sketched on either side of these cleared lines for a short distance into the dense woods. The topographer then cruises the country between these stadia lines by running compass lines (usually at right angles to the stadia lines) at, say, 100 to 300-ft. intervals, starting from one stadia line with the elevation determined by stadia level, and running at right angles through the woods to the other line, sketching contours on a small hand sketching board as he goes, using Locke's level or a clinometer for determining elevations and checking these elevations as well as position at the other stadia line ahead.

"Then he returns on the next range to the first stadia line, obtaining as he does so all the topography over the route which he has cruised. Those data are then transcribed directly on to the plane-table sheet which has been left on the first cleared stadia line. He then proceeds to the next pair of ranges, running, say, south on one and back north on the other, checking out at each time on his

levels, his distance, and on his position up and down on the stadia line as he reaches the clearing where the stadia line is run. This method gives an independent control on all the topography. It gives a check on all the cruising work, and it is the most economical method I know of for obtaining fairly accurate contours in heavily wooded districts. The transcription of the contours drawn on the sketching board to the plane-table discloses any apparent error, which should be corrected immediately in the field."

*Surveys of Cemeteries.*—As to cemetery surveys Professor Breed again had a substitute method to offer. This he did in a series of explicit directions:

"First, select a system of triangulation, the vertices of the triangles being the outer corners of stones marking cemetery lot corners. These triangles can be, say, 400 to 800 ft. on a side. Measure these distances with a tape. Plot them on the plane-table sheets by use of beam compass and scale. Use these points for the control of the detailed survey to follow. The plane-table is set up at the corner of one of these stone bounds, oriented by sighting on another triangulation point. Several other existing easily recognized monuments are then "cut in". The plane-table is then taken to another triangulation point and these same points are "cut in" and thus located on the plan. This does not require sending a man to the monument for the sight.

"Then an iron rod three times the size of an ordinary lining pin is driven in the ground at the plane-table station. This iron rod is driven through the tape ring. The tapeman measures the distance from this pin to the various lot corner markers and the plane-table man "cuts in" the direction with the alidade. He then scales on each line the distance called off by the tapeman and draws the boundary lines of each lot and records upon them the lot, its serial number, and the family name.

"Two men are all that are needed in this field party. They can survey a large number of lots in a day in this fashion. It is well not to extend the measurements from the plane-table beyond, say, 80 to 100 ft. Then the plane-table should be moved to another point which has been located from the last point. At any time the plane-table man can "cut in" his position from the many monuments which appear on his sheet, and thus check his position. In fact, it is well for him when he starts to "cut in" three or four characteristic monuments within his sheet limits.

"The next step is to measure with the tape the four sides of each lot and the two diagonals. These are recorded to the nearest 0.1 ft. on the plane-table sheet. The plane-table sheet is on the scale of 1 in. = 20 ft."

*Office Work.*—These records, remarked Professor Breed, were kept for use on a rainy day. Individual lots are plotted on large cards in card-catalogue form, the upper half being cross-sectioned and the lower portion in lines and columns. Their use is as follows:

"In the upper portion the lot is plotted to the scale of 4 ft. to the inch, or 8 ft. to the inch. These are plotted from the four dimensions and the two diagonals which were recorded in the field.

"In the lower portion of the card the tabulation of names, dates of birth and death, etc., are recorded. These are copied from the cemetery records.

"The card is then taken into the field, and at each individual lot the headstones, footstones, and monuments are located by tape measurements and ranges. Each grave is identified from the list of interments in the lower portion of the card and from wording of markers, and if there are interments that cannot be found that are indicated in the list, or graves that appear on the lot which are not in the list, further investigation is made to determine who the person is that is buried at that spot, or whether or not there is any interment."

The full advantages of this method as enumerated by Professor Breed are that: (1) It is controlled by the measured triangulation, and detail of every lot is drawn while that lot is before one's eyes; (2) it requires only two men in the field party; (3) it is economical; and (4) it presents plenty of work for rainy weather as well as for fair days.

*The Aerial Survey.*—In the property line surveys for real estate plans, Professor Breed had found aerial maps (or contact prints) useful for two appropriate purposes: (1) For reconnoissance; and (2) for filling in details or for identifying details. He continued:

"In aerial work of this sort I have found it advantageous to place cheese-cloth markers at points which I wanted to identify readily in the contact prints where the object was one that probably would not show up well on the print. These places can be readily marked by 3-ft. strips of cheese-cloth laid at right angles over these points or laid in the form of a circle around the points. Such little white crosses or circles show up clearly in the photographs.

"*Identifying Property Lines.*—The analysis of evidence of property lines is an excellent practice. In wood lots these should properly show the position of blazes marking property limits; and, where different blazed lines are encountered on the same general property line, the apparent age of the blazes should be determined by cutting into the trees and counting the annual rings, and these facts should be noted on the analysis so that the line adopted may be supported by the strongest evidence found.

"While the edge of timber clearings which usually shows up clearly in contact prints is some evidence of property lines, it should be supported by other evidence, as a rule, for it is not uncommon to find that when timber has been cleared an encroachment on to adjacent property has been carelessly made, for which damage may or may not have been adjusted."

## GENERAL DISCUSSION

The scope of the general discussion turned to the relative merits of stadia and plane-table methods. F. H. Kingsbury, M. Am. Soc. C. E., defended the use of the stadia method, because of weather conditions. Mr. Hammond declared that although the plane-table was his favorite instrument for such work, he chose stadia because the field personnel was more familiar with that method.

Professor Breed replied that he did not recommend straight plane-table work for the Swift River surveys, but a combination of field surveying with sketching tables. Only the shot through clearings in the woods would be taken by alidade observations.

## LEVELING FOR TUNNELS AND RESERVOIRS FOR THE BOSTON METROPOLITAN DISTRICT WATER SUPPLY

By Leroy G. Harris,* Esq.

The base for the levels run on the Boston Metropolitan District Water Supply Project was established, in connection with the U. S. Coast and Geodetic Survey, by running a series of "precise" or "first order" levels from

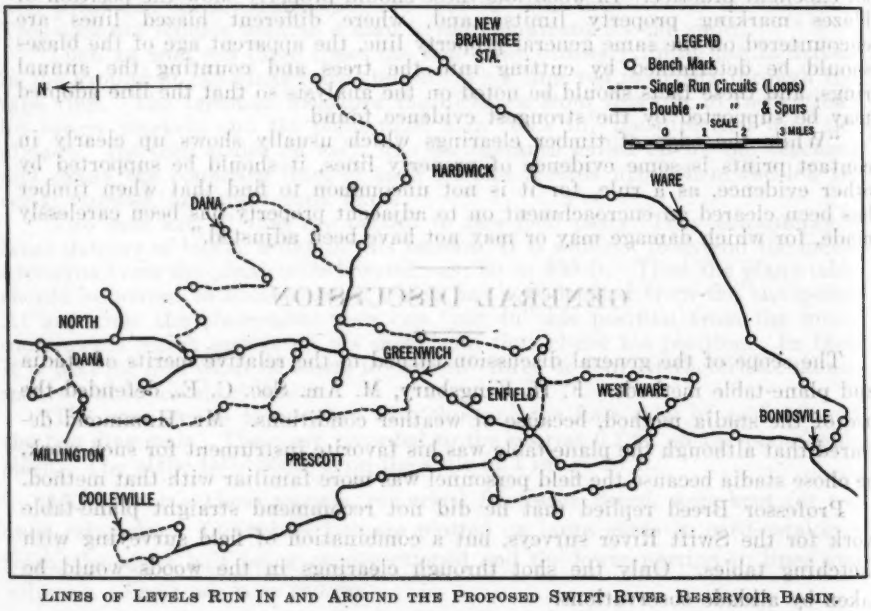
* Asst. Engr., Metropolitan Dist. Water Supply Comm., Barre, Mass.

an established Government bench-mark at Springfield, Mass., along the railroads to Boston *via* Oakdale, Mass. This established a base line from which circuits and spurs were run in and around the proposed Swift River Reservoir Basin and to the various shaft sites along the tunnel line from the Wachusett Reservoir to Swift River.

*Bench-Marks.*—The following rules were followed strictly:

Permanent bench-marks were established at intervals of approximately 2 miles. The line of levels between two bench-marks was broken by temporary bench-marks into sections from 3 000 to 4 000 ft. long, except where special conditions, such as the extremely hilly country which was encountered on some of the cross-country runs, made shorter sections advisable.

The line of levels is broken in this way in order that an error between two permanent bench-marks can be located without re-running the entire distance of 2 miles. This saves considerable time in case an error is made, especially in hilly country where progress is comparatively slow.




LINES OF LEVELS RUN IN AND AROUND THE PROPOSED SWIFT RIVER RESERVOIR BASIN.

No bench-mark was established and designated as permanent unless it was so fastened in rock, brick, or concrete as to resist extraction, change of elevation, or rotation.

At places from which it is likely that future leveling will be extended, at least three bench-marks should be established within a radius of about ½ mile, or far enough apart not to be affected by the same disturbing cause or causes.

*Leveling.*—Except when specific instructions are given to proceed otherwise, all lines are to be leveled independently in both the forward and backward directions. It is desirable that the backward measurement on each section should be made under different atmospheric conditions from those which occur

on the forward measurement, and this is usually accomplished by making the backward run in the afternoon if the forward run is made in the forenoon, and *vice versa.*

If the atmospheric conditions seem to remain the same throughout the day, the observer should secure as much difference between the forward and backward runs as is possible without materially delaying the work for that purpose.

On all sections on which the foward and backward runs differ by more than the accepted allowable error, which is generally $0.02 \sqrt{K}$ (in which, $K$ is the number of miles), the section should be re-run until any two runnings in opposite directions check within the limits of the formula.

The length of sight is usually determined by the topographic and meteorological conditions, but should never exceed 400 ft.

The length of sight between the forward and backward stations should be kept as nearly equal as possible by pacing.

When it is expected that the forward and backward runnings of the line are to be completed up to any one place, the elevation at that place should be held by two temporary bench-marks, established at least one set-up of the instrument apart. When the leveling is continued from or to such a pair of points, the instrument should be set up between them and rod readings taken on each point. The same arrangement of points should be used at the completed end or ends of any detached portion of the line of levels. Either one of the two points may be used for carrying along the elevation, with the other used only as a check against mistakes in reading the rod, or a disturbance of one or both of them. By this method, no mistake of a foot made in reading the rod, held on a bench-mark, will escape detection.

The party consisted of five men: Chief of party, observer, recorder, two rodmen, and an umbrella man. The instrument used was a Berger 3-wire Wye-level. The Molitor rods were used, and these rods have shown, after various tests, that no rod corrections were necessary. Steel turning points were used at all times, except when running along railroad tracks; then common tenpenny wire nails were driven into the ties and the head was used as a turning point. The steel turning point was driven into the ground with the aid of a hammer. After the rod readings were obtained as the back-sight the rear rodman pulled his steel pin and moved forward.

*Bench-Marks.*—The bench-marks used were of two types, one being a ¾-in., round, brass rod, 6 in. long, rounded on one end. The shank of the rod was threaded for a length of 3 in. These rods were grouted in large boulders, ledge outcrop, and concrete bridge abutments.

The other type of bench-mark was an iron rod about 5 ft. long and bent on one end. A hole approximately 18 in. in diameter was dug in the ground for a depth of 5 ft., and there filled to a depth of at least 18 in. with concrete. The rod was embedded in the concrete in a vertical position, bent end down. After the concrete had set, at least 24 hours, a piece of 2-in. pipe was set around the rod, resting on the concrete, so that the top of the rod was about ½ in. below the top of the pipe. A cap prevented the admission of water, and the hole around the pipe was refilled with earth.

All levels for this work were run during the summer months. It is not generally considered good practice to run precise levels during freezing weather as all tripod legs are fitted with metal points which absorb heat and cause more or less settlement when set on frozen ground. Then, again, it is practically impossible to drive steel pins into frozen ground and steel plates are not wholly dependable as they will also move if left too long on warm days.

The umbrella is used only when the sun is shining brightly enough to cast a shadow. The idea of the umbrella is to keep the sun's rays from affecting the instrument by expansion. When the instrument is being carried from one station to another it should be protected from the sun by covering it with a cloth or canvas hood.

*Equi-Distant Sights.*—When leveling cross-country in hilly territory it is practically impossible to keep the instrument equi-distant from the back-sight and fore-sight, but the recorder should keep the observer well informed as to these differences so that he can equalize them as soon as possible. If the fore-sight is, say, 100 ft. longer than the back-sight, then the observer, on the next set up, should shorten up on the fore-sight enough to correct this difference, without any expenditure of time for this particular purpose; so at the end of the run the total lengths of the back-sights should approximately equal those of the fore-sights.

The back-sight is generally taken first and the observer reads all three cross-hairs beginning with the top hair. They are read in feet, tenths, hundredths, and thousandths, and immediately recorded. The fore-sight is read likewise as soon as possible after the back-sight to avoid any possible error due to settlement of the instrument. The wire intervals are immediately worked out by the recorder, and if they do not differ by more than 0.006, which was selected as a maximum, the instrument was moved ahead to the next station. If the wire intervals differed by more than 0.006, the readings were taken again. On some jobs the wire interval is allowed to vary with the length of sight from 0.004 to 0.010, but they were kept low on this particular work for more or less of an experiment, and the final results showed that the error of closure and the percentage of re-running was kept comparatively low, without sacrificing speed.

*Progress.*—The speed in leveling is greatly dependent upon the activity of the observer, the topography of the ground covered, and the meteorological conditions.

The following tabulation shows the speed attained on this work and is based on levels run in and around the proposed Swift River Reservoir Basin. It does not include levels run along the main line from Springfield to Boston:

| | |
|---|---|
| Working days | 45 |
| Average working hours per day | 6.75 |
| Total mileage (including "re-runs") | 135.0 |
| Average number of miles per day | 3.0 |
| Maximum number of miles any day | 7.0 |
| Average number of miles per hour | 0.44 |

The average divergence in the forward and backward runs between adjacent benches on double-run lines was 0.0085 ft., giving an average error of

closure of $0.009\sqrt{K}$. The average error of closure for single-run circuits (loops) was $0.01\sqrt{K}$. Of these 135 miles, 7 miles, or approximately 5%, were re-run.

## DISCUSSION

### COMPARISON WITH CASCADE TUNNEL

By Henry B. Alvord,* Assoc. M. Am. Soc. C. E.

Comparison of results obtained on surveys for the Cascade Tunnel on the Great Northern Railroad furnished the subject for this discussion. This tunnel was dedicated January 12, 1929. It is 7.78 miles long, and saves 7.68 miles in distance. The maximum grade is 2.2%, and the sharpest curve is 6° as compared with 10° on the longer line.

Professor Alvord showed a number of slides illustrating the surveying problems. The topographic map of the project was drawn to a scale of 1:125 000 with a contour interval of 100 ft.

The errors of closure on the survey work for the Cascade Tunnel were given, as follows:

East Portal to Mill Creek (2 miles):

| | |
|---|---|
| Alignment | 0.23 ft. |
| Elevation | 0.20 " |
| Distance | 0.90 " |

Mill Creek to West Portal (5.3 miles):

| | |
|---|---|
| Alignment | 0.64 ft. |
| Elevation | 0.78 " |
| Distance | 1.00 " |

### CO-OPERATION OF THE UNITED STATES COAST AND GEODETIC SURVEY

By J. B. Babcock, 3d,† Assoc. M. Am. Soc. C. E.

In the beginning of his paper Mr. Harris mentioned briefly the co-operation between the U. S. Coast and Geodetic Survey and the Metropolitan District Water Supply Commission in connection with these precise levels. Some additional data are presented by Professor Babcock with regard to precise levels in Massachusetts. Regarding his own experience, he stated:

"In 1923, the U. S. Coast and Geodetic Survey ran a line of precise levels connecting Providence, R. I., Boston, Mass., and Portland, Me.‡ This leveling,

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† Prof., Railway Eng., Mass. Inst. Tech., Cambridge, Mass.
‡ *Journal*, Boston Soc. of Civ. Engrs., September, 1924.

connected with previously determined points in the base net of the United States at three places—Yonkers, Poughkeepsie, and Rouses Point, N. Y. The Boston Society of Civil Engineers was requested to co-operate in bringing city officials, public utilities, railroads, etc., in touch with the local representatives of the U. S. Coast and Geodetic Survey. On that route through Boston, the levels were connected with the railroads, the Navy Yard at Charlestown, the dry dock at South Boston, and the City of Boston levels. The Cities of Lawrence, Haverhill, and Lowell were also tied into these precise levels.

* * * * * * *

"With this line of precise levels between Providence, Boston, and Portland, the eastern part of Massachusetts was well provided with levels. Back in 1823, the so-called Van Orden line of levels was run from Boston to Albany, N. Y., under the auspices of the Massachusetts Topographical Survey through a co-operative scheme between the States of Massachusetts and New York. Unfortunately, the levels of that portion of the line in New York State have not been found, and there is no record of any tie point, which would indicate how precise those levels were. It has been the desire of the U. S. Coast and Geodetic Survey to reproduce that line at some time using the Boston and Albany Railroad as the base line; but funds have not been available to carry out this project."

However, when the Metropolitan District Water Supply Commission proposed to make surveys for tunnels and reservoir sites, it was thought advisable to have a co-operative arrangement between the U. S. Coast and Geodetic Survey and the Water Supply Commission. In this way precise levels could be run for the twofold purpose of providing the accurate datum needed by the Commission, and of establishing a backbone in an east-west direction across the State which could be tied into by the principal cities. Proceeding, Professor Babcock explained that,

"The levels were started at Springfield from a previously established U. S. Coast and Geodetic Survey bench-mark, and the line was run by this co-operative method to Oakdale, near the Wachusett Reservoir. These levels furnished the bench-marks needed for the work of the Water Supply Commission. The U. S. Coast and Geodetic Survey continued from Oakdale to Worcester and thence *via* the Boston and Albany Railroad to Riverside to a tie point with 1923 precise levels. I believe the U. S. Coast and Geodetic Survey also ran a line north from Springfield to connect with its system at St. Johnsbury, Vt.

"This has resulted in providing a definite backbone from Springfield to Boston *via* the Massachusetts Central Railroad (Boston and Maine System) and thence to Worcester and Boston, and a line from Providence to Boston and Portland, tying in the principal cities.

* * * * * * *

"The Boston Society of Civil Engineers has urged the cities and towns and the public utilities in Massachusetts to tie their level datums into the U. S. Coast and Geodetic Survey mean sea-level datum which extends from the Atlantic to the Pacific Seaboard and from the Gulf of Mexico to the Canadian border. It is recognized that in most cases it is not practicable for a city to change its long established datum, but it is of the greatest importance that the differences in datums be known. The direct use of the bench-marks of the U. S. Coast and Geodetic Survey, in connection with large-scale projects involving a number of cities and towns, will prove a practical and an economical method of leveling and one in which accuracy is assured."

## GENERAL DISCUSSION

*Inaccuracy of Leveling Rods.*—Henry G. Avers, Assoc. M. Am. Soc. C. E., commented on the small errors of closure obtained by Mr. Harris. Some of the trouble that occurs in leveling has been due to inaccuracies in the graduations on the rods.

*Coastal Surveys in Rhode Island.*—Leon L. Holland, M. Am. Soc. C. E., stated that all engineers should be obliged to use the standard datum fixed by the U. S. Coast and Geodetic Survey. The Rhode Island State Board of Public Roads has set permanent bounds all over the State, based on the Coast and Geodetic Survey Station in Providence.

*Proper Monumentation.*—The Chairman, C. H. Birdseye, M. Am. Soc. C. E., called attention to lack of co-operation in Baltimore, Md., where, at one time, there were nine separate datums.

"An equally important phase of that question", according to Chairman Birdseye, "is proper monumentation of level bench-marks with due regard to displacement from frost or other causes of disturbance, and it will interest any engineer who has anything to do with leveling to know that both the Coast Survey and the Federal Board of Surveys and Maps have issued standard specifications for leveling operations and for the monumentation of the level bench-marks, with which it would be advisable for any engineer engaged on a leveling project to acquaint himself thoroughly."

*Instruments for Precise Levels.*—Chairman Birdseye also recommended the gradual replacement of the Wye-level, or first-order level (as it is called in the Government Service) with the prism level. It was his expressed hope that some one would design "a light, easily portable, and still accurately operated prism level, so that the levelman can see the bubble at the time that he sees his rod."

## TRIANGULATION IN MASSACHUSETTS AND ITS CONNECTION WITH LAND COURT REGISTRATION

By Richard K. Hale,* Assoc. M. Am. Soc. C. E.

This paper is a compact account of the history of triangulation in Massachusetts. Mr. Hale states, in detail, the methods used, as well as recommended methods. Full credit is given to the pioneer work of Simeon Borden.

*Early Surveys.*—The towns of the Commonwealth were ordered, in 1794 and in 1830, to file with the Secretary of State plans showing the natural features and the boundaries of land owned by the State. In 1830, a surveyor was appointed to make a general survey of the entire State and this was the beginning of triangulation in Massachusetts.†

* Associate Commr., Dept. of Public Works, Div. of Waterways and Public Lands, Boston, Mass.

† Tables of the Astronomical and Trigonometrical Survey, Rept. of the Secretary of State, Commonwealth of Massachusetts, 1846.

Concerning surveying activities prior to 1794, Mr. Hale states:

"It is, of course, well understood that there were no Federal public lands in the Thirteen Original States. Those lands which had not been granted to individuals, were retained by the States when the Constitution was adopted, and such land was either granted or sold under authority given by the State Legislature. Previous to 1785, under the Massachusetts Bay Colony, the Court granted lands or plantations to groups of men called proprietors who, in turn, set off land to individuals. The remaining land came to be called common and undivided land. The title to some of these lands is still the cause of much litigation. Many references are found where committees were authorized to make surveys and lay out locations, but there is no evidence that any system of surveying was adopted or that any of the maps mentioned are available, except such partial plots as are found on file in the registries of deeds. The rectangular system of dividing territory into townships and sections as adopted by Congress in 1784 in laying out the United States Public Lands was not in vogue in the Thirteen Original States previous to 1785. Early descriptions and surveys were based upon the topography supplemented by the use of the compass and chain and the monuments used were generally watercourses, mountains, trees, and houses."

The general survey begun in 1830 was completed at a cost of about $70 000. The base line, 7.3882 miles, was in the Towns of Hatfield and Deerfield and was measured with apparatus constructed by Mr. Borden. The results were said to have been so accurate as to check, in two measurements, within 0.25 in. The ends were marked by crosses in copper bolts set in rock.

In attempting to connect the town surveys with the general triangulation many were found to be so inaccurate that much extra field work was necessary. The latitudes and departures of hundreds of points, each referred to the center of origin in one of five sections in the State, are given in the original report of this survey. Mr. Hale stated that there is no evidence of these points having been used in recent years, and that many engineers do not even know of their existence.

*Later Triangulation.*—Subsequent history was outlined, as follows:

"In 1844 the United States Coast Survey measured a base line on the location of the Boston and Providence Railroad. It was 10¾ miles long, the northerly end was in the southeastern part of Foxboro and the southerly end in the southern part of Attleboro.

"From 1844 to 1847, using this base, several primary stations were established in Eastern Massachusetts. Since then and up to the present time, the U. S. Coast and Geodetic Survey has established hundreds of triangulation stations along the sea coast, along the shores of the Connecticut, Merrimac, and Taunton Rivers, and on the islands of Martha's Vineyard and Nantucket. Many of the points used by Borden were later brought into the network established by this survey. The report of the U. S. Coast and Geodetic Survey for 1894 (Appendix 10) gives the location and geographical position of all points so determined within the Commonwealth at that time."

Continuing his outline, Mr. Hale stated that, in 1884, the Massachusetts Topographical Commission was appointed to work in conjunction with the U. S. Coast and Geodetic Survey to make a topographical map of the State. Further developments were as follows:

"During the years 1895-1898, using the primary triangulation stations of the survey as a basis, the Commonwealth established a network of triangulation extending the entire length and breadth of the State. The geographical

position of these triangulation stations were accurately determined and form the basis for all the triangulation surveys made since that time. In connection with this work, under a legislative act, between 1885 and 1914 all the angles in the boundary lines of the cities and towns were connected with the triangulation system and their geographic positions and the azimuths and distances between the angle points computed. The results of this work have been published in sixty-eight volumes in atlas form, and copies are filed at the State House, and copies of the atlas for any locality are filed in local Public Libraries and town offices. The geographical co-ordinates of the town boundary stones as tabulated give the latitude and longitude values to the nearest 0.01″ (0.01″ of longitude is 0.75 ft., and 0.01″ of latitude is 1 ft.) These bounds should not be used for triangulation points without first having their values re-computed to three places of decimals."

*Angular Measurements.*—Concerning precision, Mr. Hale observed that,

"The accuracy of determining the geographic positions of these stations by a network of triangulation is due to the method used in measuring the angles and the care and accuracy with which the signals are built and plumbed up over the stations.

"The signals are made of poles from 12 to 25 ft. high, marked with a large white flag at the top, and banded with alternate black and white stripes about 2 ft. in length. There are as many bands as the length of the pole will permit.

"The method used in measuring the angles in the triangulation system, is as follows: The instrument is clamped at random, usually near zero, and both verniers are read with great care. This reading is recorded as the setting. The first angle is read and recorded as the rough angle. The angle is repeated six times and the cumulative angle read and recorded. This reading is the mean of both verniers, and, after the setting has been subtracted, is divided by 6, giving the accurate angle. For convenience, call this the angle between Stations *A* and *B*. In the same manner, the closing angle between Stations *B* and *A* is measured. This will close the horizon, and the sum of these two angles should be, of course, 360 degrees. This horizon should close within 5″ and the error of closure should be equally divided between the two angles. If the angle is of great importance or very small, this process should be repeated several times and the average of all the observations taken as the true angle. In many large triangles, the angles are repeated in several sets of six pointings each. Sometimes as many as sixty pointings have been made for a single angle. In small triangles, one combination of angles of six repetitions each is usually sufficient to give excellent results. To offset instrumental error, the best results are obtained by inverting the telescope after every three pointings. Sometimes the angles are measured in combinations of three, *A* to *B*, *B* to *C*, and *C* to *A*, dividing the horizon error by 3 and applying the correction equally.

"In triangles where sides are 5 miles long or more, the closure error should not be greater than 3 seconds. In triangles where sides are from ½ mile to 5 miles, the closure should not be greater than 10 seconds. In smaller triangles, the error of closure may be as great as 15″ and the accuracy of the work be sufficient.

"All three angles of a triangle should be measured when practicable."

*Unused Data.*—The most important objective in providing triangulation survey data uniformly over large areas, is to furnish local surveyors with fixed points of beginning and convenient means of checking their work.

"It is amazing", said Mr. Hale, "how little use is made of this vast and costly work by those having in charge the preparation of surveys and plans of public works, especially when data must be filed in various registries describ-

ing legal takings made by public authority. There is no doubt but that Mr. Borden had in mind that the points established by him would be used from time to time in the making of all public surveys and that eventually they would be used on private surveys.

"The faulty descriptions in land conveyances, obliteration of monuments, inaccuracy of many surveys and plans, and large conflagrations which destroy not only lines but records themselves, are subjects often described in technical papers and heard discussed by engineers interested in land surveying. Their conclusions all seem to be about the same and, while they recognize the weak points in our present system, no attempt is made to correct and improve them."

*Surveys near Boston.*—The City of Boston laid down the foundation of a system of rectangular co-ordinates about 1891, using the State House as an origin, and by triangulation and traverses surveyed most of Brighton and Dorchester and filed plans showing the results.

"It is unfortunate" said Mr. Hale, "that this work was suspended and not carried over the whole city so that all its public streets and lands [old and new], could be referred to this system. * * *

"The State Department of Public Works has used this co-ordinate system, based on the State House, in its surveys in Boston Harbor, and the harbor lines as fixed by statute are given in rectangular co-ordinates, but it has not used this system in conveying or taking land, or in laying out its streets in individual areas.

"The Metropolitan District Commission uses a rectangular system based on the State House in its surveys and takings for parks.

"The Town of Swampscott has an accurate co-ordinate survey referred to an origin within the town, and, by relocations, all its old streets are now properly determined by co-ordinate values, bearings, and distances, and all new streets laid out are referred to this same system and all points properly monumented. It is doubtful if any other town within the Commonwealth has accomplished as much so far as surveys of its public properties are concerned. The City of Worcester is now working on an extensive co-ordinate survey of the city. Many towns have been surveyed on this basis for the preparation of assessors' plans and, while the values of the triangulation points and traverse stations have been computed for the purpose of making an accurate plotting, the work is never carried to the extent that co-ordinate values are used in the descriptions of new streets or takings of public lands.

"The rectangular co-ordinate system came into wide use during the World War on battle maps. Various systems were used in Europe, but in this country a zone system has been adopted, based on the polyconic projection. The zones are 8° wide in longitude, and each zone has its origin of co-ordinates. On all the U. S. Geological Survey maps recently published, the rectangular co-ordinates are shown marked in red on the margin of the sheet. This system is described in the U. S. Coast and Geodetic Survey *Special Publication No. 59.*"

*The Massachusetts Land Court.*—This Court was created in 1898. In addition to establishing the ownership of the land it determines the boundary lines which thereafter remain definite and fixed and are not disturbed by adverse possession within the meaning of the law.

"Thus," quoting from Mr. Hale, "all precautions are taken to secure an accurate survey and plan supplemented by sufficient monuments to enable all lines to be replaced at any time. This plan, called the filed plan, is made by a private surveyor under instructions approved by the Court and costs are paid by the petitioner in each case.

"The Manual of Instructions issued to surveyors sets forth briefly what is required and if at first the information furnished is not sufficient then addi-

tional data are forthcoming by order of the Court. These surveys and plans are such that when the decree of the Court is issued based upon the approved plan, called the registry plan, there is shown on the plan itself or filed with the records in the Engineering Department sufficient data to determine the location of all lines. In almost all cases traverse tables are on file showing bearings and distances with latitudes and departures computed; if not, data are available so that these calculations may be made. Where the survey is not connected with any recognized origin or initial point an origin within the survey is assumed and contiguous surveys connected with it. This is carried on until the group of surveys is, in turn, joined to some local origin connected with the State Tringulation System and then refigured when sub-divisions occur. Until this is done each survey is referred to a system of rectangular co-ordinates based on an assumed origin conveniently located.

"An accurate geodetic survey establishing triangulation points, supplemented by precise traverses compensated for error, and trigonometrical co-ordinate values assigned to stations within certain districts located near enough together so that they could be used for local work would form a complete and accurate skeleton upon which to connect registered land and would prevent embarrassing accumulations of errors, gaps, and overlaps, and furnish one origin from which all could be reproduced. The expense of such a survey is prohibitive, but with the use of such triangulation points now established and additional points established from time to time as authorized under Chapter 223, Acts of 1915, the work is gradually being carried forward and sectional plans are being prepared showing thereon registered lands and control points."

*Aids for Surveyors.*—In consequence of the Court's requirements that these triangulation points be used when within or near land being surveyed, and of requests made directly to the U. S. Coast and Geodetic Survey, the latter reported in 1915 that it would shortly publish a new book giving information on all triangulation points in Massachusetts. This book was published in 1922.* To convert geographical co-ordinates into plane co-ordinates the L.M.Z formula was first used, and later simplified by tables prepared by L. A. Chase, an Assistant Engineer in the Land Court. In 1921, the U. S. Coast and Geodetic Survey published the book, "Relation Between Plane Rectangular Co-Ordinates and Geographic Positions", giving tables for transformation of co-ordinates.† *Publications Nos. 71* and *76* should be in the possession of all engineers engaged in land survey work.

As a practical illustration, Mr. Hale described at length the solution of the three-point problem in establishing a new point by secondary triangulation for use in a local survey in the Town of Barnstable.

## DISCUSSION

### BENEFITS OF STATE TRIANGULATION

By George L. Hosmer,‡ M. Am. Soc. C. E.

*A Good Job.*—Summarizing the paper presented by Mr. Hale, Professor Hosmer declared that,

* "Triangulation in Massachusetts," U. S. Coast and Geodetic Survey, *Bulletin No. 71.*

† *Special Publication No. 71*, U. S. Coast and Geodetic Survey.

‡ Prof. of Geodesy, Mass. Inst. Tech., Cambridge, Mass.

"Massachusetts undertook to locate every town corner and every important point in the State boundaries by geodetic co-ordinates derived from triangulation, so that any lost or displaced corner could be re-set from these co-ordinates. This she succeeded in doing. The field work was completed, the calculations were made, and the main system was adjusted; and the results were published in such form that they are readily available to any one who cares to look them up."

He deplored the fact that several towns had made appropriations and had done considerable valuable surveying work, but that the result is now of no general use to engineers. Furthermore,

"Massachusetts finished her job, and did it well, and the results are available for use by engineers; but are these results being used to the extent which they should? The general opinion seems to be that they are not. If so, what is the real reason? The answer to this seems to be that there are two more things needed before the work is really complete—publicity and education.

"*Publicity and Education.*—Engineers need to be told again and again that the data of this survey are here for their use. Some of them seem hardly aware of its existence. Those who know about it may not realize its extent or in what ways it may be of use to them.

"To many persons 'triangulation' sounds highly technical and difficult. They know that they can get the data they want with transit and tape, without studying up a new subject, such as inverse solutions, so they hesitate to attempt to base their work on that of the State. There should be a little campaign of education to show how simple it really is to convert latitudes and longitudes into plane ($x$, $y$) co-ordinates with which every one is familiar.

"If engineers and surveyors could be reminded of the existence of this survey and told in a few printed pages the simple methods of using the results, more time and money might be saved, and the accuracy of surveys increased.

* * * * * * *

"*The Three-Point Problem.*—In the smaller European countries (Holland and Denmark, in particular), extensive use is made of the 'three-point problem'. In fact, it is the standard method. So many church spires, flag poles, and stand-pipes are located by plane co-ordinates that no better location is needed than can be obtained from these. The instrument is set at a corner of the property, two precise angles are measured between pairs of triangulation points, and then the traverse proceeds in the usual way. From the published co-ordinates of the trigonometric stations, and the two measured angles, the co-ordinates of the first station may be found. The others are derived from the traverse itself. It is quite possible that our State triangulation could be more often used in this manner.

"*Accuracy.*—When connecting surveys with the State triangulation there is one matter that should be carefully considered. Triangulation is accurate at the start, but the accuracy necessarily falls off as the distance from the base increases. Triangulation that has been checked by a new base or by the closure of a loop, and the angles adjusted, is certain to be accurate within the required limits; but in a survey like that of Massachusetts, there are likely to be many spur lines, not connected with other triangulation. In these, there has been no opportunity to detect the error and to distribute it. In some localities the engineer may find himself dealing with two such spurs, which have approached each other, but never have actually been connected. In this case he gets the full amount of the error of both lines, and the triangulation in this locality may prove to be weaker than the main triangulation and

perhaps less accurate than the measurements he himself is making. It is important to observe how the positions were obtained and whether they have been sufficiently checked.

"Finally, there is one benefit to be derived from a general use of the State triangulation, regardless of the accuracy obtained; that is, that all surveys are referred to a true meridian, and not to a jumble of magnetic meridians and assumed meridians. If all surveys are tied into the State triangulation, plans will then join properly instead of overlapping."

## GENERAL DISCUSSION

A publication issued by the U. S. Coast and Geodetic Survey (*Special Publication No. 71*) should be read by every city engineer, in the opinion of H. B. Alvord, Assoc. M. Am. Soc. C. E. It shows a method by which the old-fashioned inverse problem can be solved in about 15 min., or less. The method requires looking up only three logarithms and three tabular values. In the triangulation of North Adams, Mass., in 1922, Professor Alvord stated that,

"First of all I changed all of the geographical points to rectangular coordinates by this little handbook, and then all the other triangulation work was based and was done on the basis of plane trigonometry. It worked out very well."

*The n-Point Problem.*—Chairman Birdseye called especial attention to the solution of the *n*-point problem as a general extension of the three-point problem. According to Chairman Birdseye,

"It is well worth the attention of the engineer who has some facility in using graphics, because there is a phase of the problem by which you make deductions from the adopted point by use of graphics, and you can bring in five or six or ten or twelve or thirteen triangulation stations in the same solution, thereby strengthening it very much. You will find it in any of the Coast Artillery Corps publications that deal with artillery surveying, and I think in most of the publications relating to surveying that were published by the G-2 Section of the A. E. F."

H. J. Sherman, M. Am. Soc. C. E., ventured the opinion that one of the difficulties in connecting small sub-division work with existing triangulation was the element of cost. In New Jersey, all highways are being surveyed and monumented so that eventually all surveys will be connected with the State-wide system.

*Examination of Old Notes.*—Too much emphasis cannot be placed on caution in the acceptance of old notes upon which to build new work. For example, one discusser reported having found churches that had been moved some distance, or church spires that had been rebuilt off center from their original positions.

According to F. H. Kingsbury and E. W. Branch, Members, Am. Soc. C. E., the land surveyor may find that recorded triangulation stations, one after the other, have been moved by irresponsible people, or that land boundaries have

been marked by loose piles of sand. The triangulation stations of the U. S. Coast and Geodetic Survey, however, are fairly permanent in the opinion of L. L. Holland, M. Am. Soc. C. E., and it remains for the surveyor to check the work for himself.

Before closing this session of the Surveying and Mapping Division, Chairman Birdseye reported on the activities of the Society in promoting interest in the Temple Act. Prospects for action were reported as very satisfactory.

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PAPERS AND DISCUSSIONS

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# EFFECT OF TURBULENCE ON THE REGISTRATION OF CURRENT METERS

By David L. Yarnell* and Floyd A. Nagler,† Members, Am. Soc. C. E.

## Synopsis

This paper describes a series of experiments which were conducted on several types of current meters for the purpose of determining the effect of turbulence, or variations in direction, and rapid changes in speed of flow, on their registration. In contrast with experiments hitherto made by moving the meter in still water, these tests were made by holding the meter stationary in a controlled stream of water.

The results of the tests are shown in diagrams and the conclusions are given at the end of the paper.

## Introduction

The current meter is an instrument commonly used to obtain water velocities as a part of the process of making discharge measurements on natural streams, and is also frequently used similarly in open artificial channels. The meter is usually rated or calibrated by pulling it at definite speeds through quiet water. Although such a rating may be performed with extreme accuracy, it is obvious that the stream-line condition is generally quite different from the more or less turbulent flow encountered by the meter when held stationary in a moving current. In fact, parallel filaments of flow, such as are duplicated in ordinary rating methods, are seldom seen in water flowing in either natural or artificial channels. On the contrary, the main stream current often appears to be the resultant of numerous cross-currents, rotating vortices, and eddies. This is particularly true in natural channels. Less turbulent conditions exist in canals, and gauging stations may usually be located at the lower ends of long tangents where the degree of turbulence

Note.—Written discussion on this paper will be closed in April, 1930, *Proceedings*.

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is a minimum. Current-meter measurements demanding maximum accuracy often must be made in the head and the tail-races of power plants where there is likely to be considerable turbulence.

Eddies encountered in flowing streams may be considered as circling or whirling masses of water moving within the whole mass of the stream. In other words, each eddy is, in effect, a separate body of water having a momentum of its own entirely different both in direction and magnitude from that of the main current. When the impact of such a body is transmitted to the moving wheel of a current meter, its registration, for the moment, will be different from that obtained in the uniformly flowing stream.

With these eddies passing irregularly in both time and magnitude, individual velocity measurements made at a given point by a current meter may be subject to considerable variation for no other reason than that the meter was not held long enough at the point to obtain an integrated mean representing the true average current. Believing that such departures may be of a compensating nature in combining all the observed points in a given stream measurement, engineers seldom spend more than 90 sec. in securing an individual reading. Nevertheless, even in mildly eddying water, considerable variation will be obtained in two consecutive readings of the ordinary duration, and the possible departure increases with increasing turbulence.

Current meters are made in various styles, but all have a rotating element, called the wheel or propeller, actuated by the moving stream. The instrument is generally designed so that the moving wheel forms, through the proper mechanism, a contact in a compartment where the electrical terminals for registering its revolutions are placed. Thus, the number of revolutions of the wheel may be obtained by connecting the meter with an electric counter, bell, or buzzer. Some meters are constructed so that the contact is made after the wheel has turned a certain number of revolutions as 1, 5, 10, 25, or 50. Thus, from the speed of rotation as measured over a definite period of time, the velocity of the flowing water is determined.

The current meters in common use may be classified as of two types: The cup type in which the wheel rotates about a vertical axis; and the screw type in which the wheel or propeller rotates about a horizontal axis, ordinarily intended to be held parallel to the direction of flow of the stream. When a meter is held at a definite point in a turbulent stream, it may be acted upon during successive instants by different stream filaments which may be flowing in various directions and may have various speeds. Even if the current meter is held rigidly at one place, the wheel will then be acted upon, not only by stream filaments which strike the rotating element at various angles, but also by filaments with appreciable variations in speed. The combination will cause the speed of the wheel to be alternately accelerated and retarded. There has existed hitherto a dearth of information regarding the effect of turbulence in flow on the registration of different types of current meters, and it was to throw light on this question that the present investigation was made.

Some experimental studies have been made in the past to determine the characteristics of different types of meters under circumstances departing from the usually good velocity conditions found at most gauging stations; but

hitherto few data have been published showing the specific deviation to be expected under abnormal conditions in using either of the common types of meter. Nearly all canal gauging stations and many river stations have conditions that yield readings to an accuracy that is within 1 to 4 per cent. Exceptions to these stations occur where gaugings must be made: (1) From a bridge, with pier obstructions disturbing the flow; (2) in mountain streams and canals with bottoms strewn with rocks of various sizes; (3) on streams where whirls and boils with mud and sand travel slowly with the stream; and (4) where gaugings must be made along narrow crests that develop vertical augularity to the flow. At such stations the hydrographer desires particularly a meter that will be actuated by the forward components of velocity only, provided the meter equipment is such that he can hold the instrument directly up stream. If they are free to turn and face the local current, all meters, of course, will reflect the direct velocities which are greater than the down-stream components perpendicular to the gauging section. The ideal situation with parallel filaments of flow, would usually involve velocities too low to turn a meter with acceptable accuracy.

A summary of the comments of American experimenters in this field is given in the Appendix. The experience of these investigators seems to indicate that, under normal conditions such as exist at the majority of gauging stations, the current meter is to be regarded as a reliable instrument for measuring water velocity. Obviously, the best available conditions for stream gauging should always be sought. When eddies do exist, some meters have been found to register more, and others less, than the true forward component of the turbulent water. This forward component is equal to $V \cos \alpha$, in which, $V$ is the velocity of the water, and $\alpha$, the angle between the direction of local flow and the axis of the stream. Hereafter, this component will be referred to as the "cosine component".

When these experiments on current meters were begun it was expected that it would be a simple matter to put a given meter into a smooth, straight, uniform channel, carrying a known, constant, parallel-filament flow, and observe the meter reading; then produce violent turbulence by artificial means and measure the changes produced in the meter readings. Upon making the trial, however, it was found impossible to obtain consistent results that could be interpreted to any useful purpose. Therefore, it became necessary to change and expand the entire plan of experimental procedure. One step led to another until finally experiments which at the outset it had been intended to complete in 2 or 3 weeks, were carried on at intervals for 1½ years.

The experiments were made at the Hydraulic Laboratory of the State University of Iowa, during December, 1924, May and June, 1925, and May, 1926, by D. L. Yarnell, M. Am. Soc. C. E., Senior Drainage Engineer, under the direction of S. H. McCrory, M. Am. Soc. C. E., Chief, Division of Agricultural Engineering, U. S. Bureau of Public Roads. S. M. Woodward and F. A. Nagler, Members, Am. Soc. C. E., of the State University of Iowa, acted as advisers, making suggestions during the conduct of the experiments and collaborating in the preparation of the data and the report.

## Types of Current Meters Used in the Experiments

Tests were made on the eleven current meters illustrated in Figs. 1 to 8, inclusive. Several of the meters have adjustable contact points so that the meter may record various numbers of revolutions per contact. In these tests, with certain exceptions that are noted where used, the maximum number of revolutions per contact was used for each meter.

It should be noted that the Hoff meter has a rubber propeller. Since rubber, with a specific gravity of 1.3, is but little heavier than water, presumably this meter has less bearing friction and responds more readily to changes in the velocity of water than the all-metel meters. Obviously, also, the flexible rubber propeller is unlikely to be injured by contact with floating débris that might break or bend the metal propeller.

## Description of Experimental Installation

The water used in the current-meter experiments was drawn from the Iowa River above the University Dam into the Laboratory Testing Canal which is 190 ft. long, and 10 ft. wide. In the canal, below a water-tight bulkhead, was constructed a smooth concrete box-flume (see Fig. 9), 4 ft. wide, 4 ft. deep, and 36 ft. long, in which all the experiments were made. The meters were held by rigid rod suspension in the water, 10 ft. up stream from the lower end of the flume. The entrance to the flume was rounded with a 6-in. radius. For all the experiments the water flowing in the flume was kept exactly 3 ft. deep so that the cross-section of the stream was 12 sq. ft. The constant depth was maintained by means of a check-gate across the canal below the flume. This gate was a loosely fitting rectangular panel, hinged at the bottom, so that its crest could be easily adjusted to the height necessary to maintain the water surface above at exactly the desired elevation.

All the water flowing down the canal passed through the flume and was measured by a 10-ft. suppressed weir near the lower end of the canal. The weir was 8.03 ft. high and had been constructed with great care to secure maximum precision for other laboratory research, during which the computed discharge was checked by volumetric measurements of the flow. Since the cross-sections of the canal above the flume was several times as great as that of the water in the flume, the velocity of flow in the canal was low, and, in effect, the flume was drawing water from a quiet pond. The smooth rounded entrance of the flume thus produced a stream of remarkably uniform velocity, much like a jet flowing from an orifice. When there was no obstruction in the flume, measurements with a current meter at eight different points in the cross-section showed almost no measurable variation in velocity. The locations of the points and the measured values of the velocity are shown in Fig. 10.

The quantity of water entering the canal from the river was controlled by a large head-gate. Just below the head-gate is a small submerged sluice-gate which may be opened to allow water to escape from the canal back into the river just below the dam. This sluice-gate was used to make the finer adjustments in regulating the flow through the canal and flume.

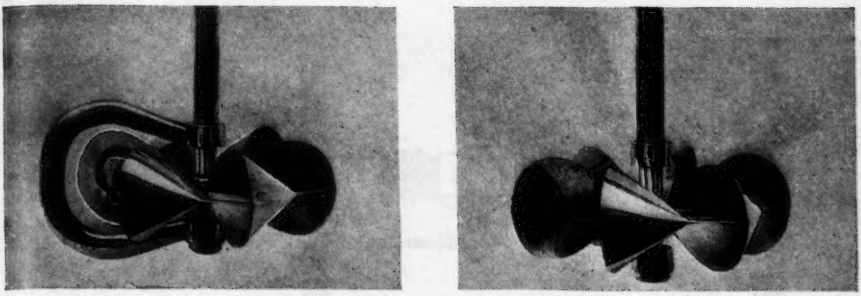


FIG. 1.—PRICE ACOUSTIC METER.

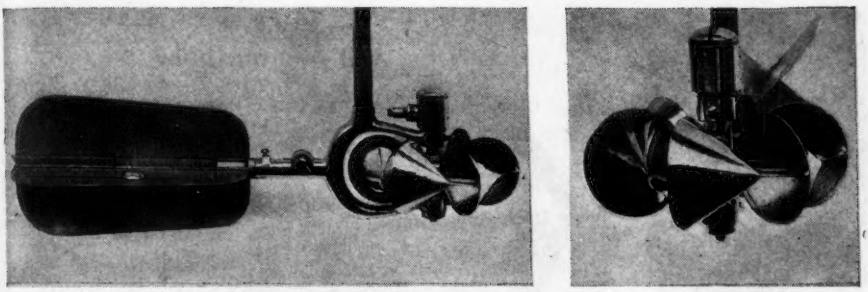


FIG. 2.—SMALL PRICE ELECTRIC METER.

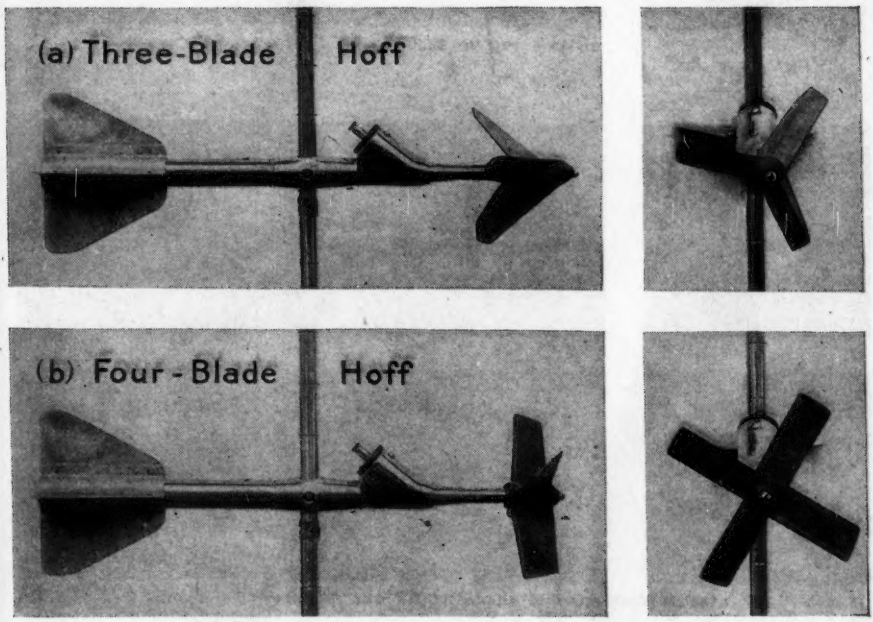



FIG. 3.—EXAMPLES OF HOFF METER WITH RUBBER PROPELLERS.

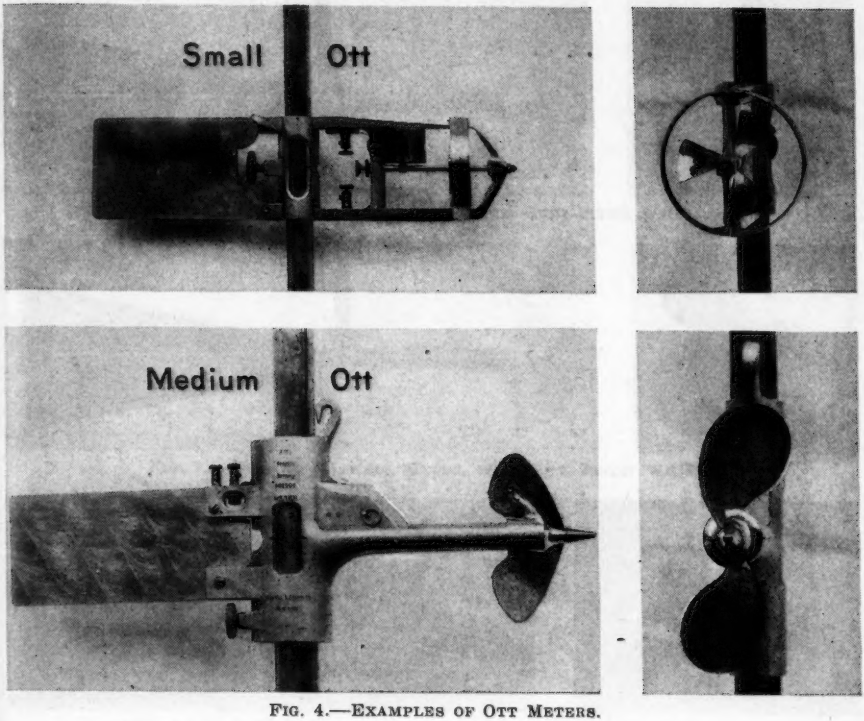



FIG. 4.—EXAMPLES OF OTT METERS.

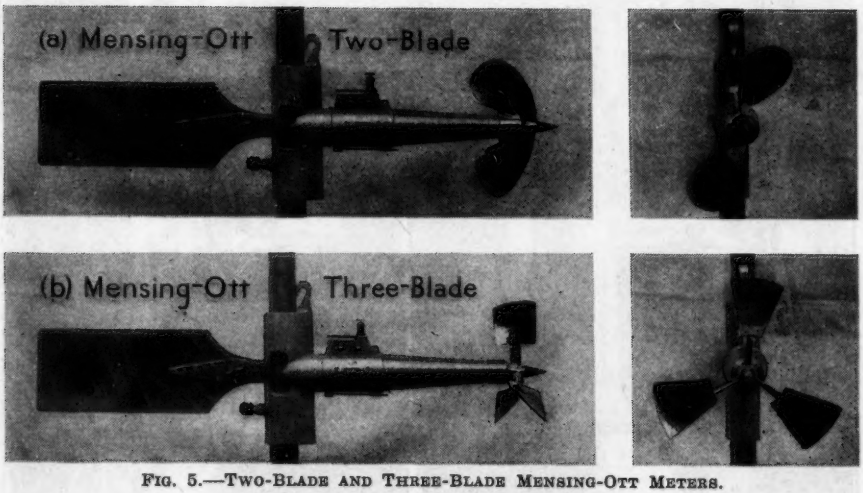



FIG. 5.—TWO-BLADE AND THREE-BLADE MENSING-OTT METERS.

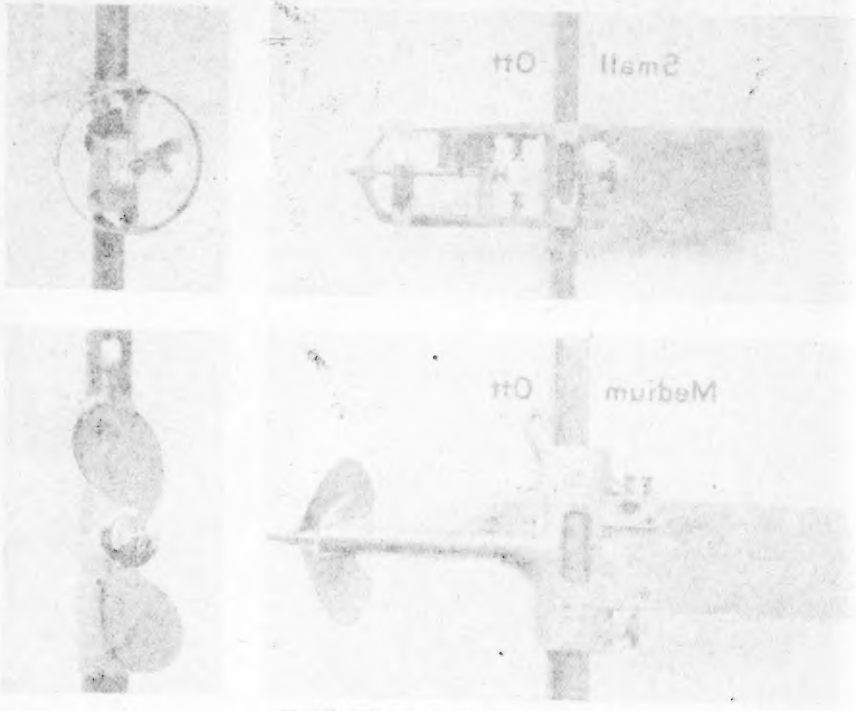



FIG. 4.—EXAMPLES OF OTT METERS.

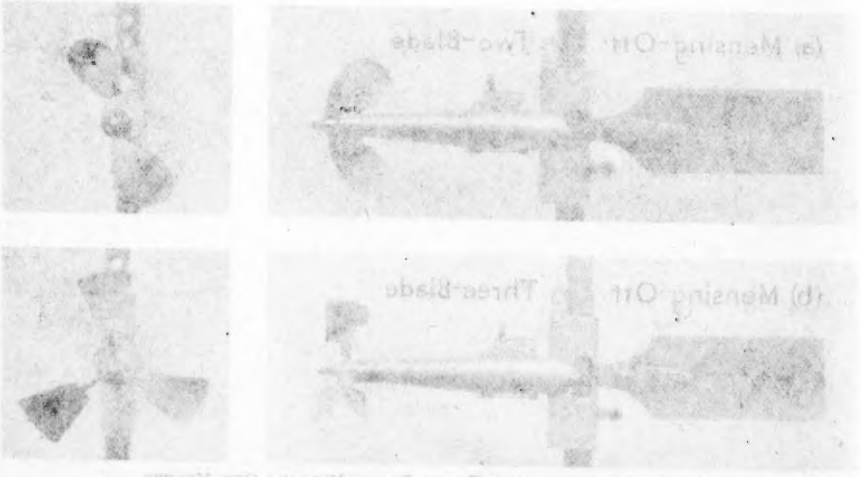



FIG. 5.—TWO-BLADE AND THREE-BLADE MENSING-OTT METERS.

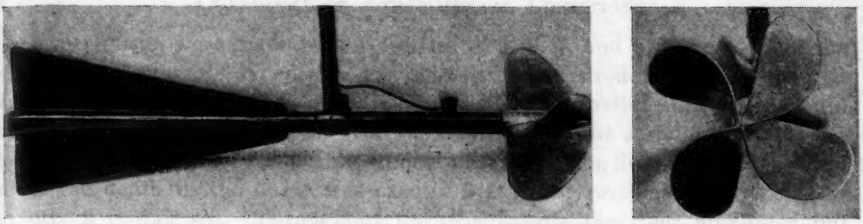


FIG. 6.—TYPE B, HASKELL METER, WITH HIGH-PITCH WHEEL.

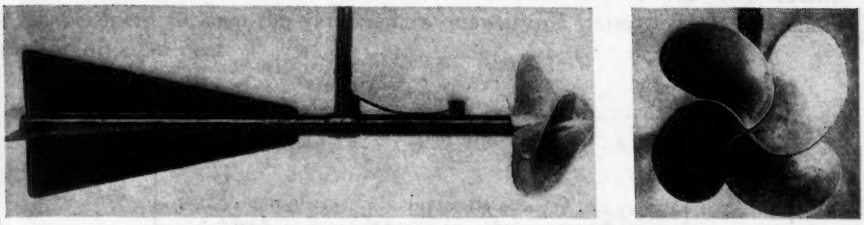


FIG. 7.—TYPE B, HASKELL METER, WITH LOW-PITCH WHEEL.

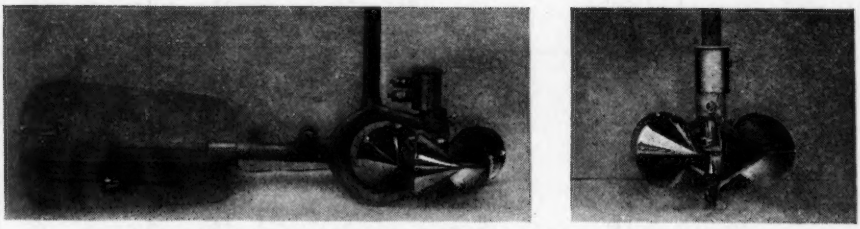


FIG. 8.—U. S. GEOLOGICAL SURVEY TYPE OF IMPROVED PRICE METER.

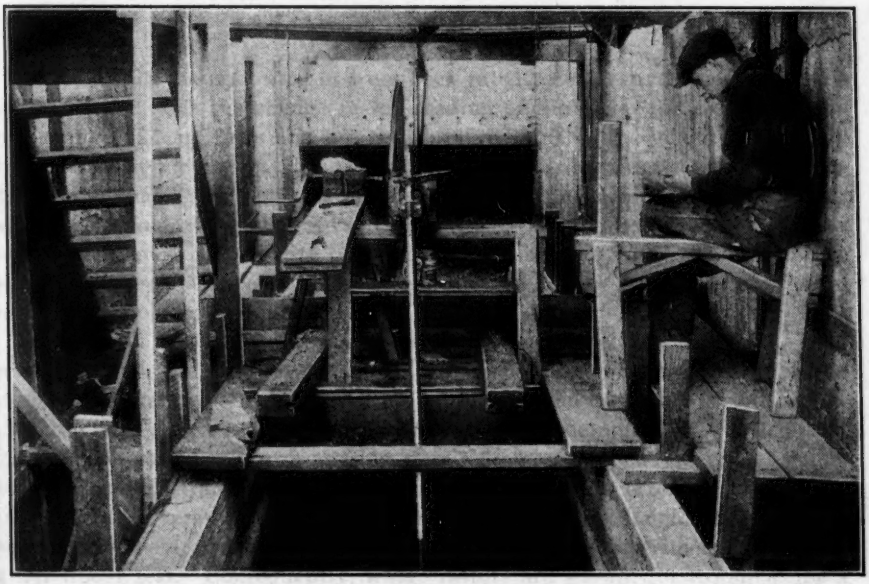
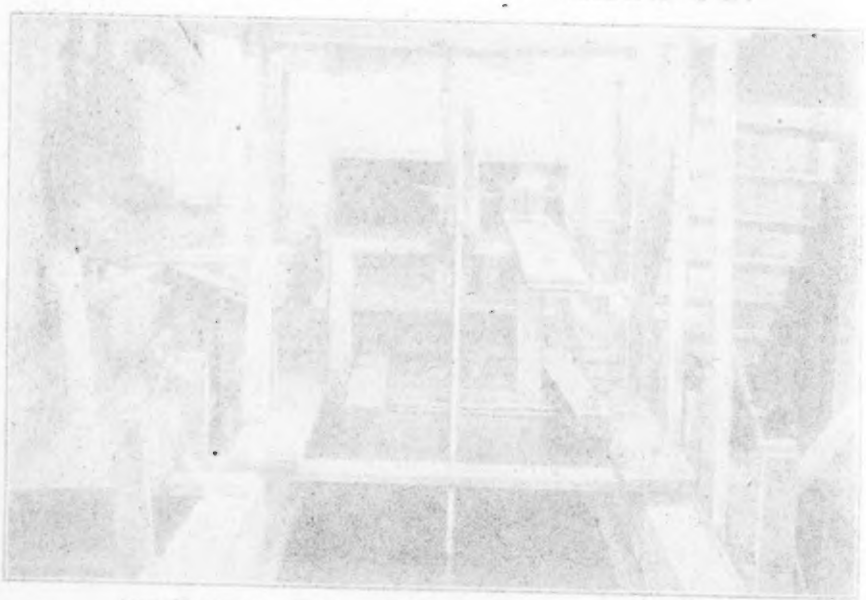


FIG. 9.—APPARATUS TO GIVE METERS VARIABLE HORIZONTAL MOTION.



## Constancy of Experimental Conditions

Experiments were made with velocities of 1, 2, 3, and 5 ft. per sec. in the experimental flume. When, for example, it was desired to run a series at 2 ft. per sec., the head-gate would be opened and the relief sluice-gate would be adjusted until 24 cu. ft. per sec. was flowing over the standard weir at the lower end of the canal. Then, the hinged gate below the flume would be raised until the depth in the flume was exactly 3 ft., thus producing a velocity of exactly 2 ft. per sec. Throughout the experiments the river fluctuated but slowly, so that when the head-gate had been once set, it was easy during any experiment to keep the variation in quantity of water flowing to a small fraction of 1 per cent.

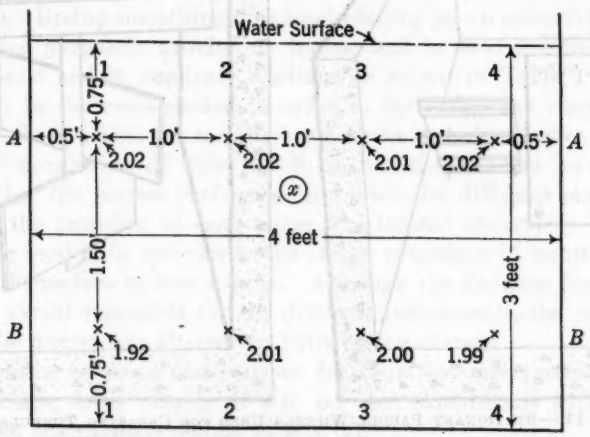



Fig. 10.—Cross-Section of Channel Looking Up Stream. Crosses Indicate Meter Positions Used in Experiments That Form First Series.

Since all the meters tested had not been rated recently, and since the study was concerned with the change in the reading of the meter for a certain definite change of conditions and not the absolute value of the meter readings, it was decided at the beginning of the investigation to compare each meter with itself only; that is, in each test the meter reading under what might be called normal or standard conditions was compared with the reading of the same meter when some abnormality of condition had been introduced. Under this plan the investigation could have been confined to one type of meter, or it could have included any number.

At the end of each day it was necessary to close the big head-gate. This gate could not easily be set with precision, the range of regulation by means of the sluice-gate was limited, and the storage in the canal was so large that after the head-gate was opened in the morning considerable time was required for the hook-gauge at the measuring weir to come to a steady constant value. This made it desirable to conserve time. Therefore, an error of 1 or 2% in any particular quantity of flow was considered close enough, and the first setting was held constant throughout the series of experiments with the meter under test. Consequently, where results are shown, for example, for a

velocity of 2 ft. per sec., the figures given for one meter may have been obtained with an actual velocity flow of 1 or 2% less than 2 ft. per sec. and with another meter with an actual velocity of 1 or 2% greater than 2 ft. per sec. This procedure was considered justifiable since, whatever actual velocity of flow obtained, it was maintained constant throughout all the experiments on one meter, and the readings obtained with one meter were compared with themselves only and not with the readings of any other meter.

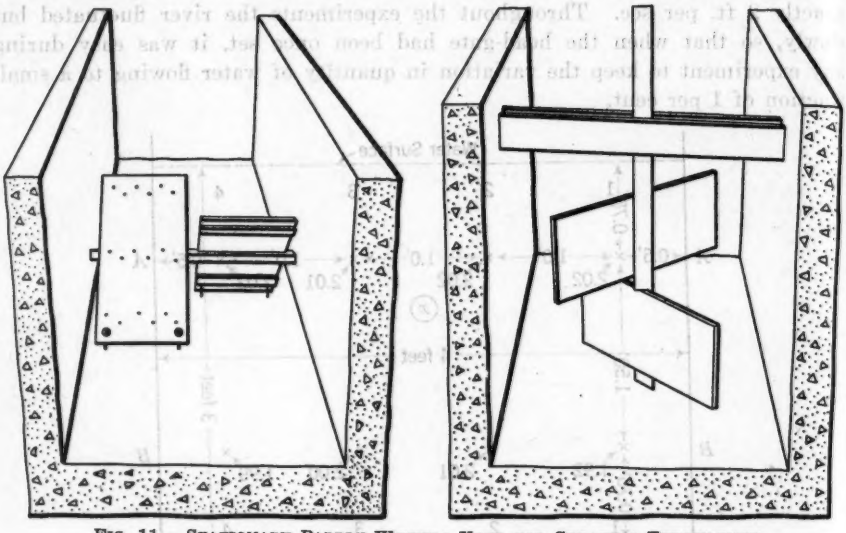


FIG. 11.—STATIONARY PADDLE-WHEELS USED FOR CREATING TURBULENCE.

## EFFECT OF TURBULENCE IN PRODUCING ERRORS IN CURRENT-METER REGISTRATION

In the first or preliminary series of experiments, in a flume carrying a constant quantity of water, varying degrees of turbulence were purposely created by the insertion of different types of baffles and obstructions. These obstructions were designed to create a definite and fixed constant disturbance in the flowing stream of water. Each meter was then held in turn at eight fixed points in the flume down stream from the obstruction, the points indicated in Fig. 10.

The fixed obstructions creating the turbulence were placed in the flume 6 ft. up stream from the testing section. In some experiments the eddies were produced by either vertical or lateral obstructions in the flume made by a submerged triangular dam, or by a vertical pier, but the most disturbed condition was obtained by the devices resembling stationary paddle-wheels, shown in Fig. 11. The disturbances set up by the stationary paddle with a vertical axis were quite different from those produced by that with the horizontal axis. The horizontal axis was placed so as to hold the lower edges of the paddles 3 in. above the floor.

The six meters used in this preliminary series of experiments are shown in Figs. 1, 2, 3 (*b*), 4, and 5 (*b*). Eight definite locations in the testing section

shown in Fig. 10 were selected for the readings. Tests were run first with no obstructions in the flume. The testing flume had rounded entrance corners and the flow conditions were more nearly ideal than ordinarily would have been encountered in natural streams, or even in artificial channels. The tests were then duplicated with the fixed obstructions in the flume.

The uniformity of velocity distribution with no obstruction in the flume may be seen in Fig. 10. The velocities shown are the average of the readings taken by four screw meters.

Since the object of this series of experiments was to determine what variation might be expected in the actual use of meters, the revolutions of the propeller were counted for a period no longer than is commonly used (generally between 40 and 90 sec.), and the time was accurately measured by means of a stop-watch. Undue smoothing of a local reading by an excessively long period of time was purposely avoided as impractical in field practice. In Experiments Nos. 8 and 9, duplicate readings as shown in Table 1 were taken at each point in the cross-section in order to determine the possibility of error by taking observations for too short a time in turbulent water.

At the conclusion of Test No. 9, an additional series was run to ascertain whether the meters performed in a radically different manner when the center of the propeller of each meter was located accurately in the plane of the testing section in contrast to the earlier procedure of locating the supporting rods themselves in this section. Although the distance from rod to meter propeller varied somewhat for the different instruments, the relative performance of the meters was altered but little by the change.

The entire series of observations for four tests are presented in Table 1. These include, for a velocity of 2 ft. per sec., experiments with the stationary paddle-like obstructions shown in Fig. 11.

When readings for the various meters are compared for the same point in the channel, it may be seen from Table 1 that in case of turbulent flow there is considerable difference. For example, at Point $A_2$ in Test No. 8, the Price meters over-registered, whereas the screw meters under-registered. In many cases the registration of the cup type of meter is more than 200% that of the propeller type for certain points in the tests with turbulent flow.

The check observations in Tests No. 8($a$) and No. 9($a$) indicate that individual observations of velocity in this turbulent water, taken for the usual length of time, may vary as much as 10 to 20%, but even this difference (which may be compensating in an average of the many observations taken in a discharge measurement), is not nearly so significant as the error of uniform sign due to the continual over-registration or under-registration of the meter in eddies and turbulent water.

While the data of Table 1 do not disclose the correct values of the average velocity of the current, it became evident from the angularity tests made later that the Price meter recorded too high a value and that the values obtained with the other meters are in varying degrees below the true amount.

Comparisons such as those presented in Table 1 demonstrate that meters may give vastly different registrations in turbulent water, but do not necessarily furnish the basis for an absolute comparison under like conditions,

since the projected area of the propellers of the different types of meters was different in both size and shape. Although the utmost care may have been exercised to locate the center of the meter wheels in identical positions, and the regimen of the turbulent condition at any point may have been uniform, it cannot be assumed that the vanes themselves encountered exactly the same kind of stream filaments since the degree of turbulence varied throughout the cross-section of the channel. These conditions prompted further methods of investigations.

TABLE 1.—COMPARATIVE CURRENT-METER READINGS SHOWING THE EFFECT OF TURBULENT FLOW ON THE REGISTRATION OF VARIOUS TYPES OF METERS.

| Test No. | Point at which meter was held. (Fig. 10.) | Measuring conditions. (Fig. 11.) | Velocity, in Feet per Second. Price Meter. Electric. (Fig. 2.) | Price Meter. Acoustic (Fig. 1.) | Hoff meter. (Fig. 3.) | Ott Meter. Small. (Fig. 4(a).) | Ott Meter. Medium. (Fig. 4(b).) | Ott Meter. Large. (Fig. 5.) |
|---|---|---|---|---|---|---|---|---|
| 8 | $A_1$ | Horizontal paddle .... | 2.71 | 2.68 | 3.01 | 2.54 | 2.71 | 2.91 |
| | $A_2$ | " " .... | 2.36 | 2.50 | 0.97 | 1.08 | 0.93 | 0.81 |
| | $A_3$ | " " .... | 2.49 | 2.89 | 1.70 | 1.48 | 1.67 | 1.38 |
| | $A_4$ | " " .... | 2.13 | 2.41 | 2.31 | 2.10 | 1.90 | 2.13 |
| | $B_1$ | " " .... | 2.36 | 2.68 | 2.53 | 2.62 | 2.70 | 2.86 |
| | $B_2$ | " " .... | 2.41 | 2.20 | 1.29 | 1.40 | 1.23 | 1.46 |
| | $B_3$ | " " .... | 1.57 | 1.62 | 0.66 | 0.62 | 0.30 | 0.40 |
| | $B_4$ | " " .... | 1.77 | 1.67 | 1.58 | 1.83 | 1.97 | 2.31 |
| 8(a) duplicate of Test No. 8. | $A_1$ | " " .... | 2.99 | 2.99 | 2.55 | 2.70 | 2.80 | 2.96 |
| | $A_2$ | " " .... | 2.33 | 2.61 | 0.96 | 1.04 | 0.91 | 0.74 |
| | $A_3$ | " " .... | 2.47 | 2.80 | 1.78 | 1.53 | 1.67 | 1.46 |
| | $A_4$ | " " .... | 2.04 | 2.42 | 2.28 | 2.17 | 1.92 | 2.12 |
| | $B_1$ | " " .... | 2.42 | 2.48 | 2.44 | 2.76 | 2.43 | 2.82 |
| | $B_2$ | " " .... | 2.57 | 2.46 | 1.40 | 1.57 | 1.06 | 1.15 |
| | $B_3$ | " " .... | 1.59 | 1.73 | 0.85 | 0.71 | 0.27 | 0.55 |
| | $B_4$ | " " .... | 1.65 | 1.55 | 1.56 | 2.05 | 1.57 | 2.32 |
| 9 | $A_1$ | Vertical paddle..... | 2.32 | 2.58 | 2.01 | 1.94 | 1.81 | 1.72 |
| | $A_2$ | " " ..... | 2.67 | 2.78 | 1.36 | 2.07 | 1.17 | 1.10 |
| | $A_3$ | " " ..... | 2.23 | 2.28 | 1.34 | 1.69 | 1.22 | 1.23 |
| | $A_4$ | " " ..... | 2.33 | 2.40 | 2.46 | 2.13 | 2.52 | 2.56 |
| | $B_1$ | " " ..... | 1.45 | 1.81 | 1.89 | 2.20 | 1.88 | 2.45 |
| | $B_2$ | " " ..... | 1.79 | 2.09 | 2.33 | 1.75 | 2.12 | 1.60 |
| | $B_3$ | " " ..... | 2.04 | 2.03 | 1.66 | 1.56 | 1.60 | 1.22 |
| | $B_4$ | " " ..... | 1.83 | 1.94 | 2.08 | 2.28 | 1.88 | 1.95 |
| 9(a) duplicate of Test No. 9. | $A_1$ | " " ..... | 2.47 | 2.78 | 1.81 | 2.01 | 1.76 | 1.61 |
| | $A_2$ | " " ..... | 2.42 | 2.78 | 1.56 | 2.11 | 1.27 | 1.28 |
| | $A_3$ | " " ..... | 2.05 | 2.40 | 1.30 | 1.69 | 1.14 | 1.16 |
| | $A_4$ | " " ..... | 2.32 | 2.41 | 2.47 | 2.14 | 2.55 | 2.56 |
| | $B_1$ | " " ..... | 1.70 | 2.08 | 1.90 | 2.13 | 1.86 | 2.43 |
| | $B_2$ | " " ..... | 2.14 | 2.18 | 2.46 | 1.63 | 1.99 | 1.58 |
| | $B_3$ | " " ..... | 2.23 | 2.00 | 1.95 | 1.64 | 1.43 | 1.18 |
| | $B_4$ | " " ..... | 1.95 | 1.98 | 1.94 | 2.28 | 1.75 | 2.01 |

Since velocity measurements were taken at each measuring point (Fig. 10) with all six meters, a comparison was made of the mean of the velocities determined at the eight points by the various meters with the mean velocity computed from the weir discharge. These data are shown in Table 2. Although a greater number of observation points would have been used if actual discharge measurements had been made, the data show certain tendencies which were verified by later tests. It will be noted that the velocities as measured

by the screw meters are usually less than their weir velocities, whereas those measured by the cup meters are generally greater.

### Effect of Fluctuations in Speed of Flowing Water

The series of experiments described, served to show that a considerable degree of turbulence in the flowing water introduced large errors into the current-meter readings; but the results did not appear to be favorable for determining what modifications of type or design in the meter itself would give most promise of reducing these errors to a minimum. Furthermore, multiplying the number of such experiments did not seem to be a worthy procedure, largely because there is no existing method of measuring degree of turbulence and no standards or units in which turbulence may be expressed. Rather than leave the matter in this incomplete and unsatisfactory state, it appeared desirable to attempt to resolve the problem into simpler elements and to attack them separately.

TABLE 2.—Percentage Variation in Mean Velocity by Meter as Determined at Eight Points from Velocity as Computed from Weir Discharge.

| Test No. | Measuring conditions. | Mean velocity in flume as determined by weir. | Percentage Variation in Mean Velocity by Meter as Determined at Eight Points, from Weir Velocity. | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Price. | | Hoff 4-blade. | Ott. | | |
| | | | Electric. | Acoustic. | | Small. | Medium. | Large. |
| 2 | Vertical obstruction.. | 1.00 | +54.2 | +53.9 | +27.0 | +49.5 | +43.7 | +44.2 |
| 3 | Horizontal paddle..... | 1.00 | +18.9 | +26.8 | −61.6 | −17.4 | −28.3 | −20.9 |
| 4 | Vertical " ..... | 1.00 | −17.4 | +18.5 | −20.2 | −15.8 | −19.9 | −14.9 |
| 5 | Submerged weir..... | 1.00 | +23.3 | +37.1 | +27.5 | +30.6 | +25.0 | +29.8 |
| 7 | Vertical obstruction.. | 2.00 | + 4.8 | + 1.2 | + 4.2 | + 6.8 | + 3.8 | −12.0 |
| 8 | Horizontal paddle..... | 2.00 | +12.8 | +30.4 | −29.5 | −12.6 | −22.2 | −15.4 |
| 8(a) | " " " ..... | 2.00 | +11.1 | +16.7 | −13.5 | −12.9 | −19.3 | −12.1 |
| 9 | Vertical paddle..... | 2.00 | + 5.1 | +13.2 | − 5.4 | − 3.1 | −13.5 | −14.3 |
| 9(a) | " " " ..... | 2.00 | + 9.5 | +13.2 | −11.2 | − 1.3 | −13.9 | −13.9 |
| 10 | Submerged weir..... | 2.00 | +38.4 | +45.7 | +30.1 | +27.3 | +29.3 | +27.0 |

When a current meter is held in a turbulent stream, the action of the water against the rotating wheel may differ in two respects, and only two, from the action in straight stream-line flow: First, the velocity of successive portions of the stream coming into contact with the wheel may vary rapidly in amount; and, second, there may be similar rapid changes in the direction or obliquity with which the water encounters the wheel. This leads to the following two kinds of errors in the results: (1) Error in registration due to the failure of the meter propeller to respond accurately by rapid changes in speed to the rapidly changing momentum of the water as the eddy passes; and (2) error in registration due to the fact that the stream lines of the eddy, as it moves down stream, strike the propeller blade at rapidly varying angles. The latter is a condition far different from the parallel filaments, flowing normal to the meter section, which propel the rotor during its rating test in still water.

Referring to the errors in the first class, a heavy propeller would be slow in acceleration and possibly equally slow in retardation. It is considered desirable that a meter shall record the correct average velocity of the passing water irrespective of its instantaneous behavior under the varying impulses of the eddy. If the meter is held at each point a reasonably long period of time, it should record this correct average.

The second series of experiments was designed to evaluate the first of these two possible sources of error in registration. Each meter, mounted on a rod supported in a frame, was immersed at a definite point (Point $x$ in Fig. 10), in a stream moving with a steady uniform straight-line velocity. The entire frame was then moved back and forth in a horizontal direction, parallel to the direction of flow at various speeds, by means of a crank and connecting rod operating through a stroke of 2 ft. Thus, instead of attempting to superimpose upon the stream of water definite and rapid changes in speed, an equivalent result was more easily accomplished by moving the meter itself, thereby producing, in the course of a test, many rapid fluctuations in the relative velocity of the water with respect to the meter. The determination of the accuracy of meters in registering the velocity of the filaments of an eddy which strike the meter at an angle was reserved for the third and last part of this investigation.

The apparatus designed to create rapid changes in the relative velocity of the water with respect to the meter is shown in Fig. 9. The large wheel could be rotated at any desired speed, so as to cause the cross-head holding the meter rod to move back and forth through a stroke of 2 ft. in a definite manner approaching a simple harmonic motion. The connecting rod, however, was only 2 ft. long; hence, the motion of the piston was unsymmetrical and the piston attained a maximum velocity somewhat greater than that of the crank-pin. Although the operator attempted to rotate the wheel at a uniform speed, probably the down-stream stroke was completed in a slightly shorter period than the up-stream stroke, since greater resistance was encountered in moving the meter against the current, and there was very little fly-wheel effect in the mechanism.

All the meters illustrated in this paper except that in Fig. 1 were attached in turn to the apparatus and tested in four different quantities of flow, giving average velocities in the channel of 1, 2, 3, and 5 ft. per sec. Under each of these flow conditions, the meters were held 1 ft. below the surface of the water in the center of the channel, and the mechanism was rotated at speeds of 5, 10, 15, 20, 25, 30, 40, and 50 rev. per min.

Each period during which the propeller revolutions were counted lasted for at least 5 min. The times of intermediate contacts were also recorded, in order to make sure that the conditions were uniform and that the meter was functioning properly.

A complete set of observations on each meter under each condition of uniform velocity was made in five steps, as follows:

1.—The desired velocity in the flume was obtained by means of the weir.
2.—An observation was made with the meter held stationary.
3.—Observations were made at each of the rotating speeds mentioned, with occasional intermediate readings with the meter held sta-

tionary in order to check the uniformity of the local velocity at the fixed point in the channel.

4.—A final observation was made with the meter held stationary.

5.—Readings on the weir hook-gauge were taken before and after each observation to check the uniformity of the discharge through the channel.

The ratio of the average revolutions per second of the meter when oscillated to the corresponding speed of rotation when the meter is held stationary in the same current is a measure of the degree to which the changes in relative water velocity affect the meter. The percentage of error in the registration of each of the meters thus obtained, plotted against the ratio of the crank-pin velocity to the uniform stream velocity is shown in Fig. 12. The points plotted are the weighted averages for stream velocities of 2, 3, and 5 ft. per sec. When any points coincide, the symbols are moved to one side of the point.

Since the crank-pin velocity is almost equal to the maximum velocity of the cross-head operating the meter, the latter ratio is not only proportional to, but almost expresses, the ratio of the maximum internal velocity of the assumed eddy to the uniform velocity of the stream.

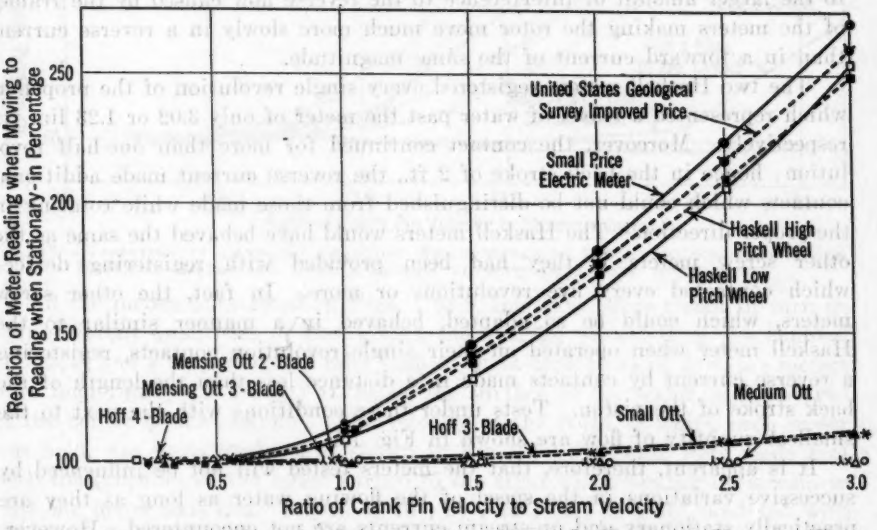



FIG. 12.—VARIATION IN RATIO OF READINGS WHEN METER IS MOVING TO READINGS WHEN METER IS STATIONARY. (WEIGHTED AVERAGE FOR STREAM VELOCITIES OF 2, 3, AND 5 FEET PER SECOND.)

Fig. 12 indicates that the two Price meters and the two Haskell meters registered excessive velocities after the crank-pin velocity had approached the stream velocity in value, whereas the remaining screw meters showed little or no effect due to the oscillation of the meter. Below this point, where no reversal occurred in direction of the velocity of the water with respect to the meter, and thus the meters, at all times, were propelled by positive currents, the changes in relative water velocities produced less error in the registration of the meters. When the velocity of the meter did not exceed one-half the velocity of the water, they all behaved alike in integrating to obtain the direct average current velocity, regardless of the weight, shape, and size of the pro-

peller. These tests show in reality that an eddy striking the meter "head on" does not affect the total counts of revolutions, provided the time of reading is not too short. According to the work of Hoff (cited in the Appendix), the weight of the propeller had an appreciable influence on the rating of the meter. Any practical difference did not become apparent in the Iowa tests.

When the maximum velocity of the backward stroke of the piston exceeded the stream velocity, a reverse current propelled the meters, during a part of the time. Under the impact of this current, the propellers of the Price meters continued to turn in a positive direction, thus over-registering; but the screw meters reversed their direction of rotation and if no meter contacts were made while rotating in the backward direction, the reverse current was automatically subtracted from the excessive forward current measured during the up-stream stroke. With the proper contact mechanism there was practically no over-registration of this type of meter at any eddy speed.

Fig. 12 indicates some slight over-registration of the small Ott meter and the three-bladed Hoff meter when the maximum backward velocity exceeded twice the forward stream current. This was probably due in these two meters to the larger amount of interference to the reverse flow caused by the frames of the meters making the rotor move much more slowly in a reverse current than in a forward current of the same magnitude.

The two Haskell meters registered every single revolution of the propeller, which represented a travel of water past the meter of only 3.02 or 1.23 lin. ft., respectively. Moreover, the contact continued for more than one-half revolution; hence in the back stroke of 2 ft., the reverse current made additional contacts which could not be distinguished from those made while rotating in the proper direction. The Haskell meters would have behaved the same as the other screw meters if they had been provided with registering devices which contacted every five revolutions or more. In fact, the other screw meters, which could be so adapted, behaved in a manner similar to the Haskell meter when operated on their single revolution contacts, registering a reverse current by contacts made in a distance less than the length of the back stroke of the piston. Tests under these conditions with the next to the smallest quantity of flow are shown in Fig. 13.

It is apparent, therefore, that the meters tested will not be influenced by successive variations in the speed of the flowing water as long as they are practically stationary and up-stream currents are not encountered. However, in case these backward currents do exist, the Price meters will invariably over-register and the screw meters will apparently do the same, unless some means of detecting the contact made in the reverse direction of rotation is adopted, or a contacting period is used, which is longer than the time in which the meter is under the influence of the backward current. If such a long interval between contacts is used, all the screw meters will perform with reasonable accuracy under ordinary changes in velocity.

### Effect on Current-Meter Registration of Stream Filaments Striking the Propeller at Various Angles

The determination of the accuracy of meters in registering the velocity of the filaments of an eddy striking the meter at an angle was the object of the

third part of this investigation. As in the previous experiments, the meters mounted on rods were held at the center line of the testing section. A pointer fastened to the rod and parallel to the axis of the meter enabled the operator to set the instrument so that the current in the testing canal struck the propeller at any desired horizontal angle (Fig. 9). In testing the Price meters with the current striking the propeller at various vertical angles, the meter rod was secured to the side of the cross-head of the machine as shown in Fig. 9. It could thus be turned to make the desired angle with the vertical.

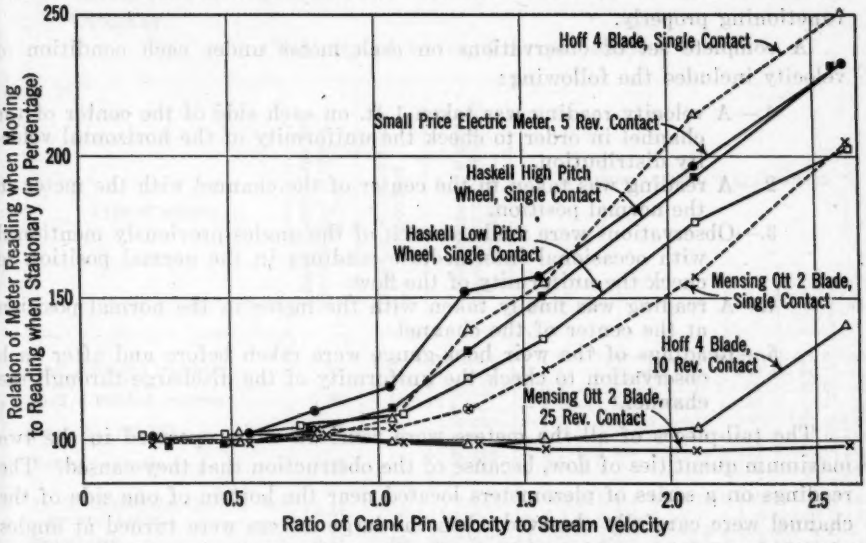



FIG. 13.—VARIATION IN RATIO OF READINGS WHEN METER IS MOVING TO READINGS WHEN METER IS STATIONARY, WITH A STREAM VELOCITY OF 2 FEET PER SECOND.

All the meters illustrated in Figs. 1 to 8, inclusive, were tested in turn in four different quantities of flow giving the average velocities in the channel of 1, 2, 3, and 5 ft. per sec. Under each of these conditions of flow, the meters were held rigidly on a rod 1 ft. below the surface of the water, with the rod at the center of the channel. The velocity at the point at which each meter was held, was quite uniform as shown in Fig. 10. The tests were made first with the propeller of the meter turned to the right when looking up stream so that the meter axis made angles of 10°, 20°, 25°, 30°, 35°, 40°, and 45° with the normal position parallel to the filaments of flow.

The meters were then tested with their propellers turned to the left through the same series of angles with the maximum quantity of flow. These readings indicated that the performance of the three Price meters (Figs. 1, 2, and 8) and the medium Ott meter (Fig. 4(*b*)) departed farther from the ideal, with their propellers turned to the left than during the previous tests with their propellers turned to the right. Consequently, these four meters were tested through the entire series of angles when turned to the left in the same quantities of flow as those used in the tests with propellers turned to the right.

Apparently, the screw meters should perform practically the same in a current striking the propeller at a given angle regardless of whether the varia-

tion in angle is in a horizontal or a vertical direction, since a symmetrical face appears to be presented to the flow. This is not true of the cup meter. Hence, the three Price meters were tested in the same quantities of flow as those used in the horizontal angle tests, but with the meter axis tilted upward 5°, 10°, 15°, 20°, 25°, and 30° and also downward through the same series of angles.

Each observation during which the propeller revolutions were counted, lasted for at least 5 min. The times of all intermediate contacts were recorded in order to make certain that conditions were uniform and that the meter was functioning properly.

A complete set of observations on each meter under each condition of velocity included the following:

1.—A velocity reading was taken 1 ft. on each side of the center of the channel in order to check the uniformity of the horizontal velocity distribution.
2.—A reading was taken in the center of the channel with the meter in the normal position.
3.—Observations were made at each of the angles previously mentioned with occasional intermediate readings in the normal position to check the uniformity of the flow.
4.—A reading was finally taken with the meter in the normal position at the center of the channel.
5.—Readings of the weir hook-gauge were taken before and after each observation to check the uniformity of the discharge through the channel.

The tail-pieces of all the meters were removed when operated in the two maximum quantities of flow, because of the obstruction that they caused. The readings on a series of piezometers located near the bottom of one side of the channel were carefully observed when the large meters were turned at angles of 45° from the normal position in order to detect any appreciable change in channel depth caused by the obstruction of the meter. The maximum difference in depth under any condition did not exceed 0.002 ft.

The removal of the 4 by 4-ft. concrete flume at the conclusion of these tests afforded an opportunity to test the meters in a channel, 10 ft. wide, at a distance 120 ft. down stream from the head-gate. Tests duplicating those previously described in which the meters were displaced through horizontal angles, were made on several of the meters at various depths of submergence. No difference in behavior from that obtained in the smaller channel was noted.

When a current strikes the wheel of a meter in a diagonal direction, the ideal meter, if prevented from turning to face the oblique flow, should register in amount only the component of the velocity of the current which is parallel to the axis of the meter and normal to the gauging section. This component is equal to the actual velocity of the water multiplied by the cosine of the angle between the meter axis and the stream line of the current.

In making the tests with the water striking the propeller at various horizontal angles, most of the meters showed a slightly different performance when the water approached the meter from the left than when it approached at the same angle from the right. The results of the tests made on all the meters using the maximum velocity of flow (the water approaching the meter from the left and then from the right) are shown in Table 3. The Price meters

over-registered every current that approached from any appreciable angle in a horizontal direction. The performance of the screw meters varied somewhat, all of them under-registering currents approaching at large angles.

TABLE 3.—READINGS RECORDED BY ELEVEN DIFFERENT CURRENT METERS, HELD AT VARIOUS HORIZONTAL ANGLES WITH THE CURRENT. RATIOS OF VELOCITY RECORDED BY METER AT VARIOUS ANGLES SHOWN AS A PERCENTAGE OF COSINE VALUE OF VELOCITY RECORDED BY METER, WITH AXIS PARALLEL TO CURRENT.

(Velocity of water, 5 ft. per sec.)

| Type of meter. | Linear distance for one revolution of propeller, in feet. | Number of revolutions per contact. | Side of meter water approaches when looking up stream. | Horizontal Angle, in Degrees. | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 10. | 20. | 25. | 30. | 35. | 40. | 45. |
| Hoff, 3-Bladed (rubber) | 0.94 | 10 | Right.. | + 1.9 | + 0.8 | — 2.7 | — 7.4 | —17.5 | —24.6 | —33.4 |
| | | | Left... | — 4.6 | —15.2 | —22.6 | —33.6 | —40.4 | —44.7 | —55.5 |
| Hoff, 4-Bladed (rubber) | 0.94 | 10 | Right.. | — 0.7 | — 5.7 | —11.1 | —16.7 | —25.9 | —37.2 | —42.7 |
| | | | Left.... | — 4.1 | —14.2 | —19.6 | —26.9 | —38.2 | —47.0 | —54.4 |
| Price, No. 211 880 | 2.23 | 5 | Right.. | + 1.7 | + 5.2 | +11.2 | +15.3 | +23.1 | +31.6 | +44.5 |
| | | | Left.... | + 1.0 | + 3.4 | + 6.2 | +10.4 | +15.9 | +21.6 | +29.3 |
| Price, Acoustic | 2.24 | 10 | Right.. | + 0.7 | + 5.5 | + 9.8 | +14 6 | +21.0 | +29.0 | +40.7 |
| | | | Left.... | + 0.7 | + 3.9 | + 6.1 | +10.3 | +15.5 | +21.5 | +28.9 |
| U. S. Geological Survey Improved Price | 2.24 | 5 | Right.. | — 0.2 | + 3.9 | + 6.0 | +12.0 | +19.2 | +30.6 | +41.4 |
| | | | Left.... | + 2.1 | + 6.2 | + 7.5 | +11.4 | +16.7 | +20.2 | +31.0 |
| Small Ott | 0.41 | 50 | Right.. | + 1.8 | + 5.3 | + 5.0 | + 3 2 | + 1.2 | — 4.2 | — 7.3 |
| | | | Left.... | + 1.2 | + 0.8 | — 2.8 | — 5.4 | —11.1 | —13.2 | —15.4 |
| Medium Ott | 0.85 | 25 | Right.. | — 2.0 | — 7.4 | —11.1 | —17.3 | —26.1 | —36.4 | —41.7 |
| | | | Left.... | — 0.4 | — 4.8 | — 8.6 | —13.7 | —20.4 | —26.8 | —34.8 |
| Mensing-Ott, 2-Bladed | 1.71 | 25 | Right.. | — 1.7 | — 6.9 | —10.9 | —11.7 | —19.9 | —20.7 | —23.8 |
| | | | Left.... | — 5.3 | —10.9 | —17.3 | —23.7 | —29.5 | —33.0 | —33.0 |
| Mensing-Ott, 3-Bladed | 1.72 | 25 | Right.. | + 0.2 | — 2.0 | — 3.5 | — 6.2 | —10.4 | —13.5 | —18.1 |
| | | | Left.... | — 2.6 | — 7.7 | — 9.1 | —11.5 | —19.3 | —24.2 | —32.6 |
| Haskell, Type B, high pitch wheel | 3.02 | 1 | Right.. | — 3.5 | —14.9 | —24.8 | —33.5 | —43.3 | —46.5 | —39.5 |
| | | | Left.... | — 4.0 | —15.9 | —24.2 | —36.4 | —45.0 | —47.1 | — 36.4 |
| Haskell, Type B, low pitch wheel | 1.23 | 1 | Right.. | — 1.6 | — 6.6 | — 8.0 | —16.8 | —24.5 | —28.4 | —31.4 |
| | | | Left.... | — 2.9 | — 9.6 | —16.4 | —20.6 | —27.9 | —32.2 | —35.4 |

When tested in the four different velocities of flow, the same meter generally showed a slight variation in performance for the same angles. The extent of the difference is indicated in Fig. 14 for one of the cup and one of the screw meters. While the variation is not large, repeated checks demonstrated that the difference observed was not accidental, but that there was a significant variation in the performance of the meters under the influence of currents of different velocities.

A weighted average of the results secured from each meter when using velocities of 2, 3, and 5 ft. per sec., was computed by giving each separate result a weight proportional to the velocity under which it was obtained, and taking the water approaching the meter from either side gave the greater divergence from the ideal. The maximum erroneous effect of oblique currents thus revealed is plotted in Fig. 15.

At large current angles, the Price meters over-registered, and all screw types registered less than the forward component of velocity.

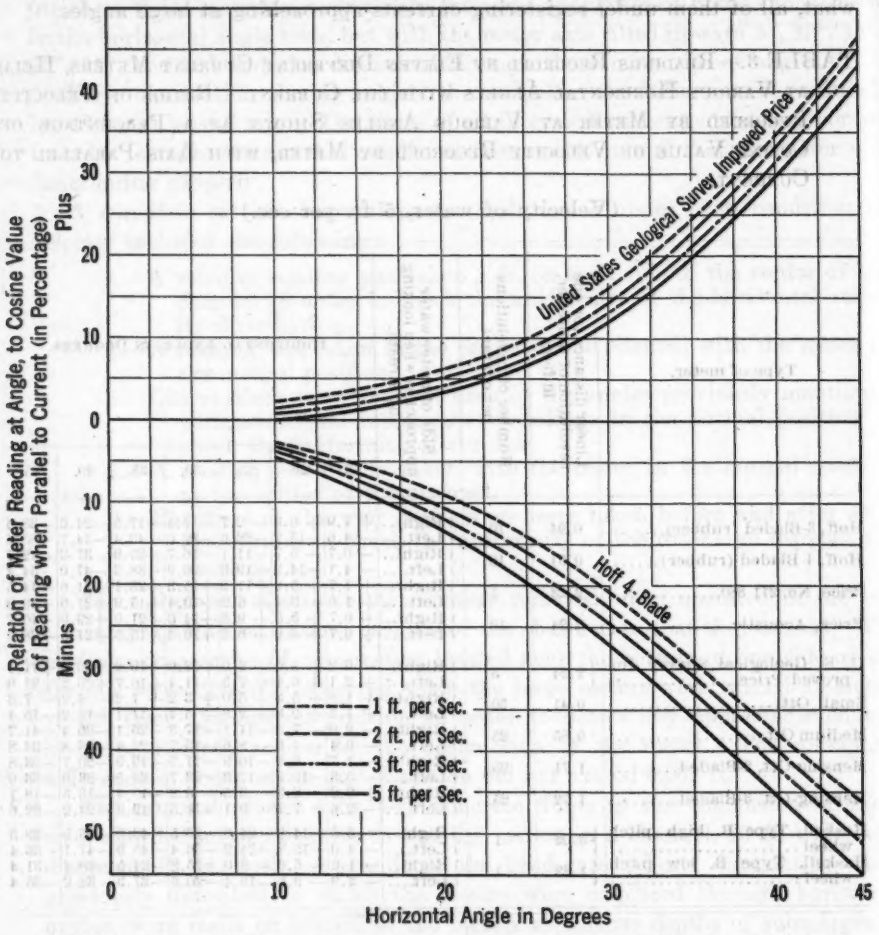



FIG. 14.—DIAGRAM SHOWING EFFECT OF HOLDING TWO TYPES OF METERS AT VARIOUS HORIZONTAL ANGLES WITH THE DIRECTION OF THE CURRENT.

The performance of the Price meters when currents approached at angles from above or below the meter (averaged for velocities of 2, 3, and 5 ft. per sec.) is shown in Fig. 16. This type of meter registers the cosine of vertical obliquities with much better accuracy than the horizontal, showing a slight tendency toward under-registration of these currents. The degree of under-registration of the component vertical to the gauging section in vertical currents, however, is not nearly as great as the amount which this meter over-registers these components in measuring horizontal disturbances.

No important difference was observed in the performance of the three types of Price meters in these tests, and probably no other type of meter will register more accurately under conditions of vertical obliquities.

In a turbulent stream, where filaments strike the meter with equal frequency from all angles, the meters would perform on the average as shown in Fig. 17. Where such a condition exists, it should be remembered that vertical and horizontal angles develop compensating effects in the cup meter. The curves shown are obtained by averaging the performance of the meter in the four directions measured in these tests. Under this condition of turbulence, the Price meter compares more favorably with the others than it did in Fig. 15.

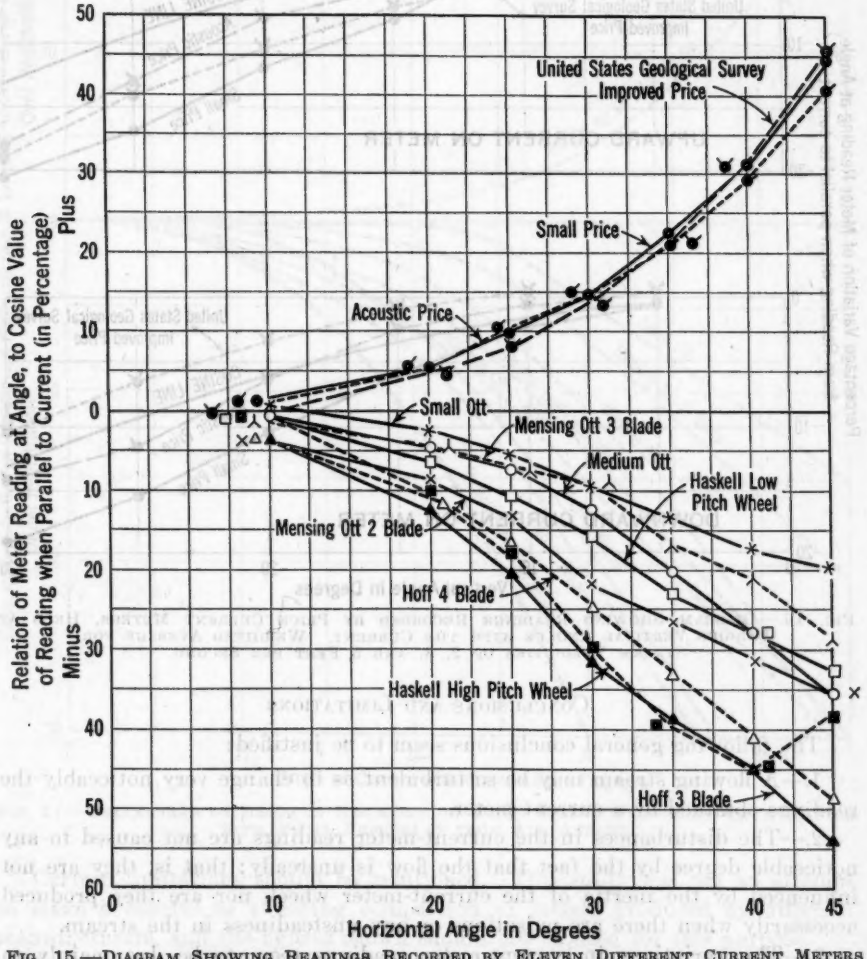



FIG. 15.—DIAGRAM SHOWING READINGS RECORDED BY ELEVEN DIFFERENT CURRENT METERS HELD AT VARIOUS HORIZONTAL ANGLES WITH THE CURRENT.

Under these conditions, the small Ott meter (Fig. 4(*a*)) would prove to be an instrument of remarkable accuracy, registering the true average component of all currents which approached the meter at angles up to 20 degrees.

In making measurements around débris, bridge piers, and side contractions which cause horizontal disturbances, the Price meter will over-register appre-

ciably; but in metering in channels where disturbances originate at the bottom of the stream, causing a preponderance of vertical currents, the Price meter held on a rod should register the desired velocity with a good degree of accuracy.

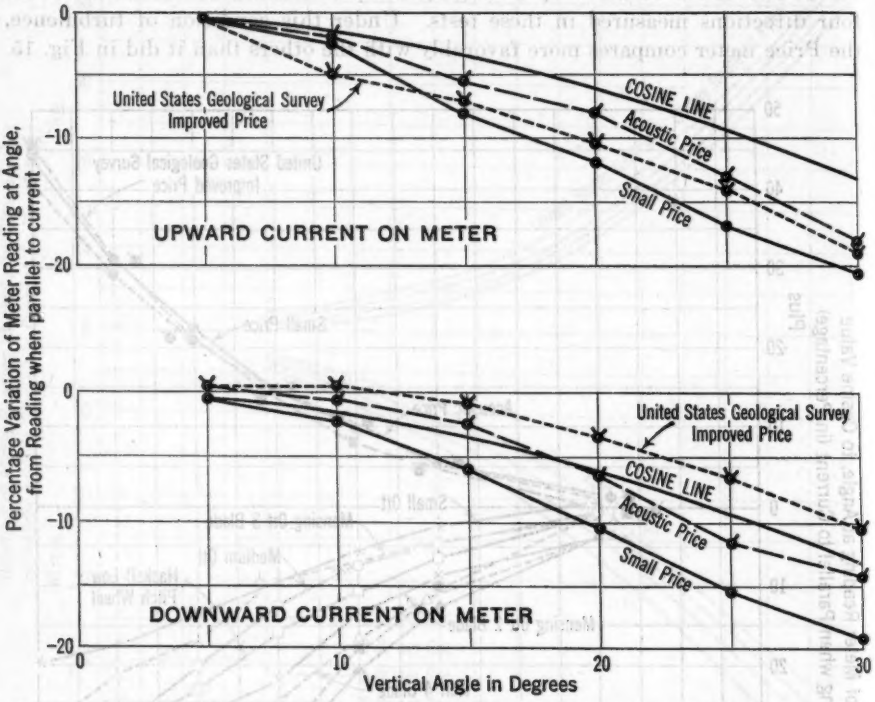



FIG. 16.—DIAGRAM SHOWING READINGS RECORDED BY PRICE CURRENT METERS, HELD AT VARIOUS VERTICAL ANGLES WITH THE CURRENT. WEIGHTED AVERAGE FOR STREAM VELOCITIES OF 2, 3, AND 5 FEET PER SECOND.

## CONCLUSIONS AND LIMITATIONS

The following general conclusions seem to be justified:

1.—A flowing stream may be so turbulent as to change very noticeably the readings obtained by a current meter.

2.—The disturbances in the current-meter readings are not caused to any noticeable degree by the fact that the flow is unsteady; that is, they are not influenced by the inertia of the current-meter wheel, nor are they produced necessarily when there are pulsations or any unsteadiness in the stream.

3.—The variations in current-meter readings seem to be due mainly to the obliquity of flow in the filaments striking the meter wheel.

4.—When meters are held on a rod in a rigid position, with the meter axis parallel to the general axis of the stream, turbulent flow invariably causes the cup meter to over-register and the screw type to under-register.

5.—The cup meter registers the true velocity of currents approaching the meter from almost every horizontal direction; hence it over-registers the com-

ponent perpendicular to the gauging section of horizontally oblique flow. Obliquity of flow in a vertical direction has a slight tendency to cause under-registration and thus may compensate to some degree for the over-registration caused by horizontal obliquity.

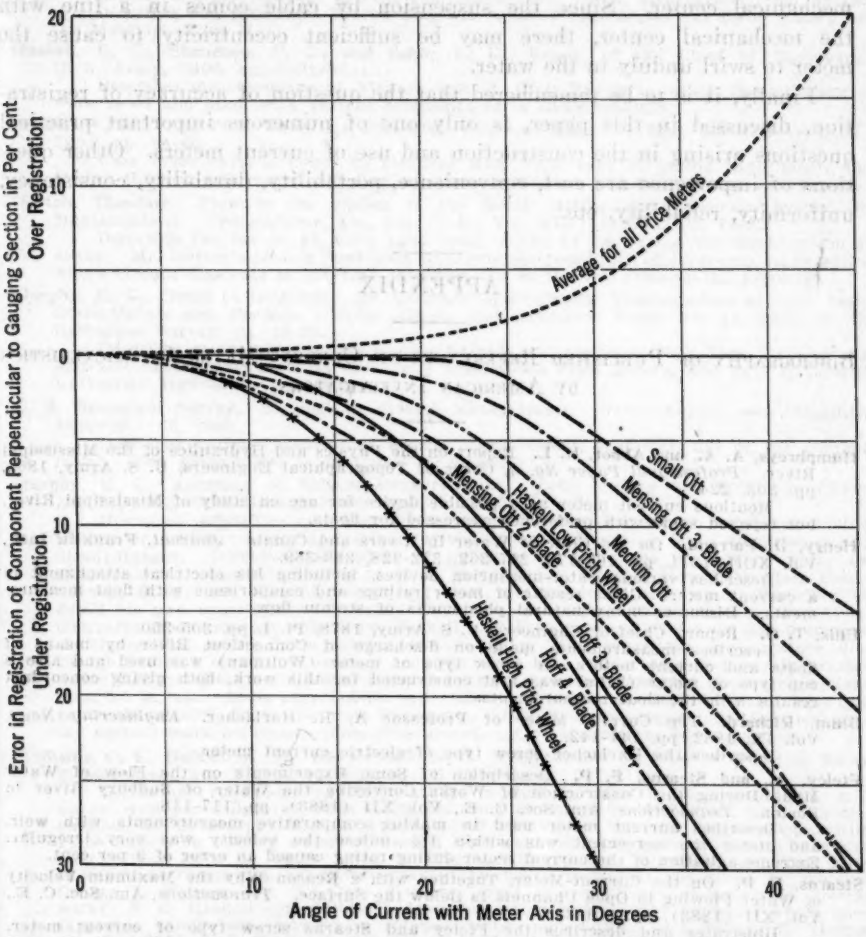



FIG. 17.—PERCENTAGE OF ERROR IN REGISTRATION WHEN STREAM FILAMENTS STRIKE METER WITH EQUAL FREQUENCY FROM ALL ANGLES.

Certain limitations should be mentioned. This study was not an attempt to make a careful or sweeping comparison of meters produced by different manufacturers, and the figures shown should not be used for this purpose.

It should be remembered also that rod support was used in all these experiments, and the conclusions must not be applied without modification to cable-suspended meters. In a majority of cases, stream gauging is done with cable-suspended meters. In such suspension the tail-vanes tend to keep the meter squarely facing the current regardless of any local or temporary obliquity of flow. In some places, however, such as close to the sharp corners of

piers or obstructions, the curvature of the filaments may be so sharp that the direction of flow at the meter wheel is appreciably different from that at the vane. In such case oblique flow must necessarily strike the current wheel. Furthermore, the dynamic center of some meters does not coincide with the mechanical center. Since the suspension by cable comes in a line with the mechanical center, there may be sufficient eccentricity to cause the meter to swirl unduly in the water.

Finally, it is to be remembered that the question of accuracy of registration, discussed in this paper, is only one of numerous important practical questions arising in the construction and use of current meters. Other questions of importance are cost, convenience, portability, durability, consistency, uniformity, reliability, etc.

## APPENDIX

### Bibliography of Published References on Current-Meter Characteristics by American Investigators

**Humphreys, A. A., and Abbot, H. L.** Report on the Physics and Hydraulics of the Mississippi River. *Professional Paper No. 4*, Corps of Topographical Engineers, U. S. Army, 1861, pp. 203, 224.
Mentions current meter as a possible device for use on study of Mississippi River, but rejected same with preference expressed for floats.

**Henry, D. Farrand.** On the Flow of Water in Rivers and Canals. *Journal*, Franklin Inst., Vol. XCII, 1871, pp. 167-173, 257-262, 322-328, 383-389.
Describes various water-measuring devices, including his electrical attachment to a current meter. Gives results of meter ratings and comparisons with float measurements. Discusses many natural phenomena of stream flow.

**Ellis, T. G.** Report, Chief of Engineers, U. S. Army, 1878, Pt. I, pp. 305-350.
Describes measurements made on discharge of Connecticut River by means of floats and current meters. A screw type of meter (Woltman) was used and also a cup type of meter (Ellis) was first constructed for this work, both giving concordant results with the float measurements.

**Blum, Richard.** The Current Meter of Professor A. R. Harlacher. *Engineering News*, Vol. IX, 1882, pp. 141-142.
Describes the Harlacher screw type of electric current meter.

**Fteley, A., and Stearns, F. P.** Description of Some Experiments on the Flow of Water Made During the Construction of Works Conveying the Water of Sudbury River to Boston. *Transactions*, Am. Soc. C. E., Vol. XII (1883), pp. 117-118.
Describes current meter used in making comparative measurements with weir, and states the agreement was within 1% unless the velocity was very irregular. Extreme agitation of the current meter during rating caused an error of 9 per cent.

**Stearns, F. P.** On the Current-Meter, Together with a Reason Why the Maximum Velocity of Water Flowing in Open Channels Is Below the Surface. *Transactions*, Am. Soc. C. E., Vol. XII (1883), pp. 301-338.
Illustrates and describes the Fteley and Stearns screw type of current meter. Gives results of rating experiments in which an irregular and variable velocity was created by oscillating the meter; and experiments during which the axis of the meter was turned at various angles with the direction of motion of the rating car up to a maximum of 41°, in which position the meters under-registered about 10 per cent. Results of twenty-seven experiments are given in which the discharge of the Sudbury Conduit was measured both by current meter and weir with varying degrees of agreement depending mostly on the method used in taking the velocity observations by current meter.

**The Price Current Meter.** *Engineering News*, Vol. XXIX, 1893, pp. 196-197.
Describes large Price meter invented by the late W. G. Price, M. Am. Soc. C. E., and patented August 25, 1885. Follows article describing current-meter measurements on Niagara River.

**Price, W. G.** A New Current Meter and a New Method of Rating Current Meters. *Engineering News*, Vol. XXXIII, 1895, p. 27.
Describes Price acoustic current meter and method of rating from skiff in still pond.

**The Ritchie-Haskell Direction-Current Meter.** *Engineering News*, Vol. XXXIII, 1895, pp. 27-28.
Describes the Haskell current meter and also direction-recording apparatus by which the direction of the current may be observed.

**Noble, Theron A.** Gauging of Cedar River, Washington. *Transactions,* Am. Soc. C. E., Vol. XLI (1899), pp. 1–26.

Describes the use of a Haskell meter in measuring stream flow, and compares same with measurements by floats. The discussion as to the accuracy of the measurements has historical interest.

**U. S. Geological Survey.** Annual Report, 21st, 1899-1900, Pt. 4, Hydrography, pp. 28-29.

Describes and illustrates by two diagrams a screw type of current meter known as the Moulinet, Type H.

**Haskell, E. E., Shenehon, F. C., and Sabin, L. C.** Report of the Chief of Engineers, U. S. Army, 1900, pp. 5322-5401.

Describes measurements of Niagara and St. Clair Rivers in which Haskell meters were rated and used, with various comments as to their reliability.

**White, W. M.** The Pitot Tube, Its Formula. *Journal,* Assoc. of Eng. Societies, Vol. 27, 1901, pp. 64-66.

Uses Price current meter to compare with Pitot tube in rating experiments by towing through still water, showing excellent agreement between the two instruments.

**Horton, Theodore.** Flow in the Sewers of the North Metropolitan Sewerage System of Massachusetts. *Transactions,* Am. Soc. C. E., Vol. XLVI (1901), pp. 78-92.

Describes the use of an Ellis (cup type) meter in measuring the discharge in a sewer. Mr. Horton's closing discussion mentions the possibility of erroneous registration where stream filaments of different velocities act on the two sides of the propeller.

**Murphy, E. C.** Tests to Determine the Accuracy of Discharge Measurements of New York State Canals and Feeders. *Water Supply and Irrigation Paper No. 47,* 1901, U. S. Geological Survey, pp. 18-29.

Describes comparative measurements by rods and meter on New York State canals and also experiments made in 1900 with rods, small Price meters, and weir at Cornell University Hydraulics Laboratory.

**U. S. Geological Survey.** Methods of Stream Measurement. *Water Supply and Irrigation Paper No. 56,* 1901.

Describes various methods of stream measurement, including use of the current meter. The Price meter is described in detail.

**Murphy, E. C.** Accuracy of Stream Measurements. *Water Supply and Irrigation Paper No. 64,* 1902, U. S. Geological Survey, pp. 11-95.

Discusses difficulties and instruments used in measuring stream velocities, giving history of current meter, describing various types, and stating their advantages and disadvantages. Discusses the effect of pulsation of velocities of moving water on measurements. Extracts results of Ellis, Marr, Henry, Gordon, New York State Canal Survey, Fteley and Stearns, comparing float measurements with weir. Describes in detail his own experiments at Cornell University and Chevy Chase, Md., in 1900-1901. Compares measurements with Price, Haskell, and Fteley meters with weir measurements. Concludes that small Price current meters under ideal conditions can measure discharge within 1 or 2%; it is more accurate when held rigidly on a rod rotating faster under this condition; will under-register current velocities closer to the water surface than 0.5 ft., etc. Some experiments under abnormal conditions showed results departing as much as 40% from the weir discharge. Haskell meter registered velocities near the surface more accurately than Price meter, not as accurate in low velocities.

**Williams, G. S., Hubbell, C. W., and Fenkell, G. H.** Experiments on Flow of Water in Pipes. *Transactions,* Am. Soc. C. E., Vol. XLVII (1902), pp. 337-341.

Occasional reference to meter ratings in flowing water as compared with still water, with particular reference to its accuracy as compared with Pitot tube. In discussion, E. C. Murphy (pp. 202-203) gives results of eighteen measurements with Price meters in moving water at Cornell University showing agreement with standard weir within 2.8% in every case; D. Farrand Henry (pp. 224-226) describes early experiments comparing velocities obtained by floats with that registered by meters; George Y. Wisner, (pp. 231-233) discusses error in registration of meters in flowing water; E. E. Haskell (pp. 283-289) describes rating tests in flowing water which gave favorable comparison between a Haskell meter and floats; and Robert E. Horton (pp. 289-291) discusses nature of eddies and the performance of current meters.

**Murphy, E. C.** Current Meter and Weir Discharge Comparisons. *Transactions,* Am. Soc. C. E., Vol. XLVII (1902), pp. 370-391.

Describes fifty discharge measurements with Haskell and Price meters in testing canal at Cornell University with agreement between meters closer than with weir, but in every case variations were less than 4.8%; twenty of the tests showed differences less than 1 per cent. In discussions: Charles H. Miller (pp. 379-380) discusses comparative results with different current meters, describing experiments with Haskell and Price meters mounted on a skiff which was rocked, producing over-registration of the Price meter and under registration of the Haskell meter; J. B. Lippincott (pp. 383-387) describes eleven experiments comparing Price and Haskell measurements, also, in some cases, with weir; mentions possible error involved in integration method using Price meter; and E. E. Haskell (pp. 387-388) expresses his opinion of the relative merits of screw type and cup type of meters.

**Murphy, E. C., Hoyt, J. C., and Hollister, G. B.** Hydrographic Manual of the United States Geology Survey. *Water Supply and Irrigation Paper No. 94,* 1904, U. S. Geological Survey, pp. 19-31.

Discusses use of meters in obtaining discharge measurements, and describes Price meters in detail.

**Murphy, E. C.** Accuracy of Stream Measurements. *Water Supply and Irrigation Paper No. 95*, 1904, U. S. Geological Survey, pp. 1-169.
Revision of *Water Supply Paper No. 64* with additional experimental data comparing Price and Fteley meters in shallow streams of varying roughness with weir measurements. The part giving data on vertical velocity curves has been augmented by additional data.

**Barrows, H. K.** Work of Hydrographic Branch, United States Geological Survey, in New England, and Discussion of Methods for Estimating Stream Flow. *Journal*, Assoc. of Eng. Societies, July, 1905, pp. 19-20.
Describes work of the U. S. Geological Survey on surface water resources in New England, including current practice in securing discharge measurements with Price meter.

**Senate Document 105**, 62d Congress, 1st Session, pp. 72-74.
Describes an experiment by F. C. Shenehon, M. Am. Soc. C. E., with a Haskell meter in the Detroit River in 1906, comparing the velocity registered by this meter with that observed from the rate of travel of liquid blueing. Good experiment was obtained.

**Gregory, W. B.** Test of a Rotary Pump. *Transactions*, Am. Soc. Mech. Engrs., Vol. 28 (1907), pp. 745-769.
Gives results of pump discharges measured in a flume by both Price electric current meter and a Tulane Pitot tube. Individual measurements showed a maximum variation of 2% between the results obtained in measuring about 153 cu. ft. per sec. with the two different instruments and an average variation of but 0.1 per cent.

**Hoyt, John C.** A Universal Current Meter. *Engineering News*, Vol. 57, 1907, p. 263.
Describes adaptability of small Price meter to stream gauging, and gives rating curves for Fteley, Warren, Haskell, small Price, large Price, and Lallie current meters.

**Stevens, J. C.** Comparison of Formulas for Computation of Stream Discharge. *Engineering News*, Vol. 59, 1908, pp. 682-684.
Discusses errors involved in various methods of using current-meter data in obtaining the discharge of a stream. Recommends the method adopted by the U. S. Geological Survey, and also multiplying each depth by its corresponding velocity.

**Hoyt, John C.** Recent Changes of Methods and Equipment in the Water Resources Work of United States Geological Survey. *Engineering News*, Vol. 60, 1908, pp. 15-16.
Describes refinements which were made in construction and use of Price meter, referring to pentacount registering device, torpedo weights, rods, etc.

**Hoyt, John C.** The Use and Care of the Current Meter as Practiced by the United States Geological Survey. *Transactions*, Am. Soc. C. E., Vol. LXVI (1910), pp. 70-105.
Gives history of current-meter development, plates showing Price, Haskell, Fteley, Ellis, Richards, Woltman, Ott, and Amster meters. The use, care, and rating of the Price meter is thoroughly discussed. In discussion, A. H. Dramont (pp. 106-107) describes gaugings and ratings of Fteley meter in measuring the discharge of Croton Aqueduct; E. C. Murphy (pp. 107-108) mentions the possibility of the different behavior of a meter near the sides and bottom of a channel; H. K. Barrows (pp. 108-110) stresses certain precautions in use of the Price meter; C. E. Grunsky (pp. 110-126) discusses the difference in results obtained in computing discharges from average of velocity observations taken at the end of sections as practiced by the U. S. Geological Survey in contrast to the method proposed by W. H. Hall, M. Am. Soc. C. E., in 1878 of multiplying velocity measurements by depth at the point of the measurement. Mentions possibility of error in meters of various types and other gauging problems; E. Kuichling (pp. 126-128) suggests various possibilities of error in the rating and registration of meters; G. H. Matthes (pp. 128-129) concludes that the precision of the current meter is far ahead of other observations made in obtaining discharge data on natural streams; Walter Pearl (p. 130) agrees with the conclusion of Mr. Matthes; and C. M. Allen (pp. 130-132) expresses confidence in measurements with the current meter, based on experiments at the Hydraulic Laboratory of the Worcester Polytechnic Institute and elsewhere.

**Van Winkle, Franklin.** Stream Flow at Single Cross-Section. *Power*, Vol. XXXII, Pt. 2, 1910, pp. 1553-1557.
Describes pressure plate, Pitot tube, float wheel, and current meters. Gives detailed discussion of Price and Ott meters. Use and advantages of each.

**Stevens, J. C.** Hydrometry as an Aid to the Successful Operation of an Irrigation System. *Transactions*, Am. Soc. C. E., Vol. LXXI (1911), p. 323.
Gives results of twelve measurements made for the purpose of comparing the small Price pentacount head with the single count head. The maximum difference was 1.8 per cent.

**Williams, Gardner S.** Measurement of Water. *Journal*, Western Soc. of Engrs., Vol. XVII, 1912, pp. 22-48.
Discusses various devices, including current meters, mentioning comparison with float measurements at Cornell University and work of U. S. Lake Survey.

**Hosea, R. M.** Measurement of the Flow of Water in Large Pipe Lines. *Journal*, Western Soc. of Engrs., Vol. XVII, 1912, pp. 34-39.
Describes special Price meter of electrical and mechanical type used on rods inside 28-in. wood-stave pipe. Rating curves for same are shown.

**Groat, B. F.** Characteristics of Cup and Screw Current Meters. *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), pp. 819-840, 852-870.
Describes tests made with Haskell and Price current meters and Pitot tube at Massena, N. Y., concluding that cup meters over-register (on the average 6%) more than screw meters under-register (the average for Haskell was about 1 per cent.). States that current-meter measurements in perturbed water should be made with

caution. Results of experiments given in which meters were oscillated longitudinally and transversely through a small amplitude in a still-water rating, and also some ratings from a skiff with oscillations caused by rocking the boat. An addenda states that experience on some mountain streams indicates that cup meters will over-register three to six times the amount which the screw meter under-registers. In discussion, W. G. Price (pp. 841-844) defends the Price meter and states its merits; E. E. Haskell (pp. 844-846) states merits of screw meter and gives eight comparative measurements between the velocities obtained by meters and floats; C. H. Miller (pp. 846-848) stresses accuracy in rating meters; and J. C. Hoyt (pp. 848-852) gives reasons for the U. S. Geological Survey adopting the Price meter and concludes that it will give reliable results at all but the most exceptional stations.

**Rumpf, Charles P.** An Investigation of the Use and Rating of the Current Meter. *Engineering News*, Vol. 71, 1914, pp. 1083-1084.

Gives results of tests on cup and screw meters holding meters at various angles with the current.

**Groat, B. F.** Pitot Tube Formulas, Facts and Fallacies. *Proceedings*, Engrs.' Soc. of Western Pennsylvania, Vol. 30, 1914-15, pp. 323-383.

Paper devoted mostly to Pitot tube, but on pages 351-366 gives results of ratings of large Price and Haskell current meters from boats under ideal conditions and with boat rocking; describes use of these meters in turbine tests at Massena, N. Y. Concludes that cup meter was accelerated by rocking and screw meter retarded.

**Moody, Lewis F.** The Measurement of the Velocity of Flowing Water. *Proceedings*, Engrs.' Soc. of Western Pennsylvania, Vol. 30, 1914-15, pp. 280-323.

Paper devoted mostly to Pitot tube, but on pages 320-322, the results of comparative ratings of a Pitot tube, Price meter, and Fteley and Stearns meter are compared with the meters turned through various horizontal angles up to 70 degrees. Also gives results of an experimental screw meter with good characteristics. Concludes cup meter over-registers and registers velocities from right differently from those from left; screw meter under-registers. In discussion, Gardner S. Williams (p. 386) describes results of rating of Haskell meter by weir in turbulent water in which Haskell under-registered more than 3%; N. C. Grover (pp. 391-397) discusses work of the U. S. Geological Survey and adaptability of Price meter to this work; efforts to develop screw meter; and general history of Price meter; Thomas P. Roberts (pp. 397-400) remarks generally on some difficulties encountered with current meters; Charles M. Allen (pp. 401-402) describes Worcester Polytechnic Institute circular rating station; Clemens Herschel (p. 410) remarks generally to the effect that current-meter gauging is only adapted to the measurement of large open channels; E. H. Brown and F. Nagler (pp. 415-424) give account of tests made on large and small Price current meters operated at center of 42-in. main, rotated at various angles with the thread of the current, through 360° in horizontal plane and 180° in vertical plane. Curves for small Price meter show different registration to left from that of right, over-registration at all angles in horizontal plane, and satisfactory or under-registration for angles in vertical plane. Curves for large Price meter show its inferiority to small Price meter; Lewis F. Moody (pp. 454-455) gives results from Rensselaer Polytechnic Institute thesis on Price meter inclined at various horizontal and vertical angles up to 12°, confirming Brown and Nagler tests; and B. F. Groat (pp. 460-473) further discusses the Massena current-meter work, design of ideal meter, and mentions same having been produced at Rensselaer Polytechnic Institute. Comments on Haskell meter, and effect of bearing friction in current meters, possible inertia effect of heavy propellers.

**Scobey, Fred C.** Behavior of Cup Meters Under Conditions not Covered by Standard Ratings. *Journal of Agricultural Research*, May 25, 1914, pp. 77-83.

Gives results of experiments on Price current meter rated in still water on rod and cable, tipped varying amounts to 30° upward and downward, also near water surface, bottom, and channel sides, with dull pivot, and vertical oscillation of meter. Concludes meter should be held horizontal, channel sides have little effect, but bottom and surface readings are in error; dull pivot has inappreciable effect for velocities greater than 1 ft. per sec.; vertical meter movement rotates the meter in the same direction as moving water.

**Harding, S. T.** Experiments in the Use of Current Meters in Irrigation Canals. *Journal of Agricultural Research*, November 8, 1915, pp. 217-232.

Comparisons of the relative accuracy of discharge measurements made in irrigation canals with current meters using different methods of measuring velocities in the verticals are discussed. No type of meter is mentioned.

**Groat, B. F.** Chemi-Hydrometry and Its Application to the Precise Testing of Hydro-Electric Generators. *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), pp. 1231-1271.

Describes still-water rating experiments on Haskell, Ott, and Price meters at University of Michigan, in which the meters were oscillated at various periods through arcs of various lengths in a horizontal, vertical, and longitudinal direction. Longitudinal oscillations showed that water pulsations affect meters but little, while lateral disturbances made the Price meter over-register, whereas the Ott and Haskell meters under-registered, the under-registration of the Haskell meters exceeding that of the Ott type. In discussion, Robert E. Horton (pp. 1283-1285) discusses accuracy of current-meter measurements; and W. S. Richmond (p. 1287) mentions several comparative measurements between current meters and other methods, and instances of erroneous registration of both types of meters.

**Hoyt, J. C., and Grover, N. C.** River Discharge. Fourth Edition, 1916, pp. 6-22.

Discusses history of and use and care of current meter with particular reference to the Price meter, duplicating article by J. C. Hoyt, *Transactions*, Am. Soc. C. E., Vol. LXVI (1910), p. 70.

**Blanchard, Murray.** Hydraulics of the Chicago Sanitary District's Main Channel. *Journal*, Western Soc. of Engrs., Vol. XXV, 1920, pp. 483-506.

Describes measurements with Haskell and Price meters under a variety of field conditions with data from occasional still-water ratings made during the progress of the work. Concludes that the Haskell meters appeared more accurate and reliable than the cup meters.

**Fortier, S., and Hoff, E. J.** Defects in Current Meters and a New Design. *Engineering News-Record*, Vol. 85, 1920, pp. 923-924.

Discusses the inaccuracies of meters under field conditions and describes the new Hoff meter giving curves comparing the effect of friction in various types of meters.

**Gutmann, O.** Glorifying the Current Meter. *Engineering News-Record*, Vol. 97, 1926, p. 840.

Review of "Der Genauigkeitsgrad von Flügelmessungen bei Wasserkraftanlagen", by Dr. Ing. A. Staus, in which the propeller type of meter is shown to be more reliable than the cup type.

**Hoyt, K. K.** Defense of the Price Current Meter. *Engineering News-Record*, Vol. 97, 1926, p. 1010.

Letter defends the use of Price meter by the U. S. Geological Survey, comparing the behavior of same with screw meters advocated by Dr. Ing. A. Staus.

**Matthes, G. H.** Price Current Meter: Serviceability *vs.* Scientific Design. *Engineering News-Record*, Vol. 98, 1927, p. 126.

Letter stresses the fact that the hydrographers of the U. S. Geological Survey during the early existence of the Bureau found the Price meter to be more serviceable than other types.

**Groat, B. F.** In Defense of Current Meters. *Engineering News-Record*, Vol. 98, 1927, p. 370.

Letter defends current meter as a valuable measuring device, and mentions characteristics of different types.

**Liddell, William A.** Stream Gauging. First Edition, 1927, p. 9 and pp. 136-150.

Discusses experimental work done in the United States, the effects of turbulent flow, and the behavior of cup-shaped and screw-shaped meters when subjected to various tests.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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# GENERAL SPECIFICATIONS FOR STEEL RAILWAY BRIDGES

PREPARED BY COMMITTEES FROM
THE AMERICAN SOCIETY OF CIVIL ENGINEERS
AND
THE AMERICAN RAILWAY ENGINEERING ASSOCIATION

CONFERENCE COMMITTEES

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| American Railway Engineering Association | O. F. DALSTROM, *Chairman*,<br>O. E. SELBY,<br>I. L. SIMMONS,<br>F. E. TURNEAURE. |

MARCH, 1929.

NOTE: This Specification is published for the information of the membership, and others and discussion thereon is invited.

## FOREWORD

A conference of Committees was formed in 1925 by appointments from the American Society of Civil Engineers and the American Railway Engineering Association, to make a study of the Specifications for Steel Railway Bridges issued by each of these bodies, with a view to preparing for adoption a single set of specifications.

The American Railway Engineering Association specifications were adopted by the Association in 1920, and were published in Vol. 21 of its *Proceedings*. A number of revisions have been made. The third edition, which is the latest, was issued in August, 1925.

The American Society of Civil Engineers specifications were adopted by the Society in 1923, and were published in Vol. LXXXVI (1923) of the Society's *Transactions*. No revisions have been made.

Both sets of specifications were submitted in 1924 to the American Engineering Standards Committee for adoption as a Standard, but neither was adopted. After the discussion of the specifications before a committee of the American Engineering Standards Committee, it was suggested that the sponsor bodies appoint Conference Committees to review the specifications for the purpose of eliminating the differences. This suggestion was favorably received and Conference Committees were agreed upon, each to consist of four members of the sponsor body. The appointment of the undersigned members followed.

The Committees met in Chicago, Ill., on January 30, 1926, to organize, decide on methods of procedure, and outline the program of work. At this meeting the Committee agreed to extend invitations to Robert Farnham, M. Am. Soc. C. E., Chief Engineer, Philadelphia Terminal Improvements, Pennsylvania Railroad, O. E. Hovey, M. Am. Soc. C. E., Assistant Chief Engineer of the American Bridge Company, and Lee H. Miller, M. Am. Soc. C. E., Chief Engineer of the American Institute of Steel Construction, to attend the Committee's meetings and act in an advisory capacity. All accepted the invitations and have taken active part in the work of the Committee. Mr. Farnham was succeeded in 1928 by A. R. Wilson, M. Am. Soc. C. E., Engineer of Bridges and Buildings, Pennsylvania Railroad.

The Conference elected officers as follows: O. F. Dalstrom, *Chairman*; Henry B. Seaman, *Vice-Chairman*; and I. F. Stern, *Secretary*.

Conference Committee, American Society of Civil Engineers

- Henry B. Seaman, *Chairman*,
- C. R. Harding,
- I. F. Stern,
- George F. Swain.

Conference Committee, American Railway Engineering Association

- O. F. Dalstrom, *Chairman*,
- O. E. Selby,
- I. L. Simmons,
- F. E. Turneaure.

March 16, 1929.

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## QUESTIONS TO BE ANSWERED FOR THE INFORMATION OF BIDDERS

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Length of spans.
Types of spans.
Number and spacing of tracks.
Angle of skew.
Type of floor.
Limiting under-clearance.

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# GENERAL SPECIFICATIONS FOR STEEL RAILWAY BRIDGES

## PART I.—DESIGN AND MANUFACTURE

### SECTION A.—PROPOSALS AND DRAWINGS

**Definition of Terms.**

1. The term "Company" means the Railway Company party to the contract. The term "Engineer" means the Chief Engineer of the Company or his subordinates in authority. The term "Inspector" means the inspector representing the Company. The term "Contractor" means the manufacturing or fabricating contractor party to the contract.

**Proposals.**

2. Bidders shall submit proposals conforming to the terms in the letter of invitation. The proposals preferably shall be based on plans and specifications furnished by the Company. The plans will show the conditions determining the design of the bridge, the general dimensions, stresses, and typical details.

Invitations requiring the Contractor to furnish the design shall state the general conditions at the site, such as track spacing, character of foundation, presence of old structures, traffic conditions, etc.

**Shop Drawings.**

3. After the contract has been awarded and before the work is begun, the Contractor shall submit to the Engineer for approval, prints in duplicate of stress sheets and shop drawings, unless such drawings shall have been prepared by the Company.

Shop drawings shall be made on the dull side of the tracing cloth, 24 by 36 in. in size, including margins. The margin at the left end shall be 1½ in. wide, and the other margins ½ in. The title shall be in the lower right-hand corner.

No change shall be made on any approved drawing without the consent of the Engineer, in writing.

The tracings of the drawings shall be the property of, and be delivered to, the Company after the completion of the contract.

4. The Contractor shall be responsible for the correctness of his drawings, and for shop and field connections, although the drawings may have been approved by the Engineer.

5. Ordering of material by the Contractor prior to the approval of the drawings shall be at his risk.

**Drawings to Govern.**

6. If the drawings and the specifications differ, the drawings shall govern.

**Patented Devices.**

7. The Contractor shall protect the Company against claims arising from the use of patented devices or parts proposed by him.

**Notice to Engineer.**

8. No material shall be rolled nor work done before the Engineer has been notified where the orders have been placed.

### SECTION 1.—GENERAL FEATURES OF DESIGN

**Materials.**

101. Bridges shall be made wholly of structural steel, except where otherwise specified. Rivet steel shall be used for rivets. Castings shall be of steel unless cast iron is specifically authorized by the Engineer.

**Types of Bridges.**

102. The types of bridges to be used for various span lengths may be as follows:

Rolled beams for spans up to 40 ft.
Plate girders for spans up to 125 ft.
Riveted trusses for spans 100 ft., or longer.
Pin-connected trusses for spans 150 ft., or longer.

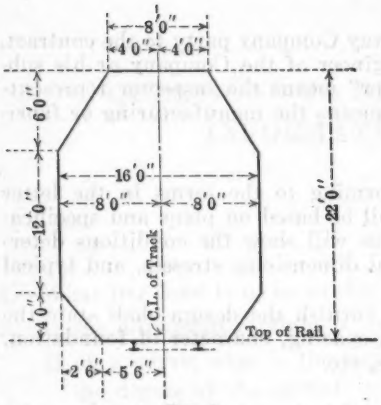



FIG. 1.

**Spacing of Trusses, Girders, and Stringers.**

103. The distance between centers of trusses or girders shall be sufficient to prevent overturning by the specified lateral forces. In no case shall it be less than one-twentieth of the span for trusses, nor one-fifteenth of the span for girders.

The girders of deck spans and the stringers of through spans shall be spaced not less than 6 ft. 6 in. between centers. If four stringers are used under one track they shall be arranged in pairs, each pair symmetrical about a rail.

**Depth Ratios.**

104. The depth of trusses preferably shall be not less than one-tenth of the span. The depth of plate girders preferably shall be not less than one-twelfth of the span. The depth of rolled beams used as girders and the depth of solid floors preferably shall be not less than one-fifteenth of the spans. If smaller depths are used, the sections shall be so increased that the deflection will not be greater than if these limiting depth ratios were not exceeded.

**Clearances.**

105. The clearances on straight track shall not be less than those shown in Fig. 1. On curved track the clearance shall be increased to allow for the overhang and the tilting of a car 80 ft. long, 60 ft. between centers of trucks, and 14 ft. high.

Unless otherwise specified the superelevation of the outer rail shall be taken as 1 in. per degree of curvature. The distance from the top of rail to the top of tie shall be taken as 8 in.

**Dimensions for Calculation.**

106. For the calculation of stresses the length shall be:

For trusses and girders, the distance between centers of bearings.
For floor-beams, the distance between centers of trusses or girders.
For stringers, the distance between centers of floor-beams.

and the depth shall be:

For pin-connected trusses, the distance between centers of pins.
For riveted trusses, the distance between centers of gravity of chords.
For plate girders, the distance between centers of gravity of flanges, but not exceeding the distance out to out of flange angles.

**Ambiguity of Stress.**

107. Bridges shall be so designed as to avoid, as far as practicable, ambiguity in the determination of the stresses.

**End Floor-Beams.**

108. Spans with floor systems preferably shall have end floor-beams.

**Skew Bridges.**

109. In skew bridges the ends of the supports for each track shall be square with the line of track.

**Floor.**

110. Ties shall be not less than 10 ft. long, and spaced not more than 6 in. apart in the clear. They shall be secured against bunching.

## SECTION 2.—LOADS AND STRESSES

**Loads and Stresses.**

201. Bridges shall be proportioned for the following loads:

*a.*—Dead load.
*b.*—Live load.
*c.*—Impact or dynamic effect of the live load.
*d.*—Centrifugal force, including impact.
*e.*—Other lateral forces.
*f.*—Longitudinal force.

Stresses from each of these loads and forces shall be shown separately on the stress sheet.

**Dead Loads.**

202. In estimating the weight for the purpose of computing dead load stresses, the following unit weights shall be used:

| | | |
|---|---|---|
| Steel | 490 | lb. per cu. ft. |
| Concrete | 150 | " " " " |
| Sand, gravel, and ballast | 120 | " " " " |
| Asphalt-mastic and bituminous macadam | 150 | " " " " |
| Granite | 170 | " " " " |
| Paving bricks | 150 | " " " " |
| Timber | 5 | " " ft. B. M. |

The track rails, inside guard rails, and fastenings shall be assumed to weigh 200 lb. per lin. ft. for each track.

**Live Load.**

203. The minimum live load for each track shall be as shown on Fig. 2, which is hereby designated as the A-64 loading.

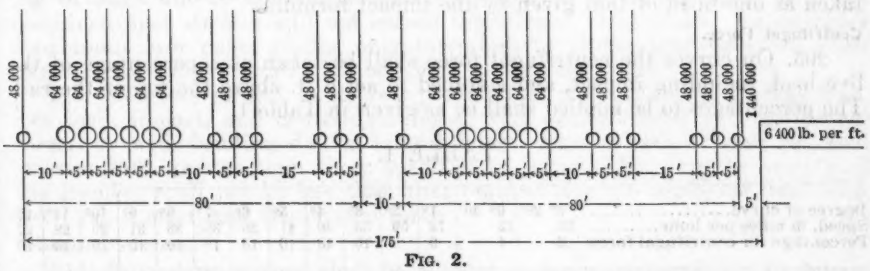



FIG. 2.

If specified by the Engineer, there may be substituted for the A-64 loading the following simplified loadings:

For spans of less than 40 ft., a series of seven axle loads of 64 000 lb. each, spaced 5 ft. between centers.

For spans of 40 ft. or more, a uniform load of 6 400 lb. per ft. of track with a single load placed in the most effective position. For moments, the single load shall be 128 000 lb.; for shears, the amount given by the formula:

$$E = 128 + \frac{1}{2} L$$

in which,

$E$ = the single load, in thousands of pounds; and
$L$ = the span length, in feet.

The results obtained by the use of these simplified loadings will approximate those from the A-64 loading.

In special locations, where conditions limit the loading to light engines, a lighter loading, as stipulated by the Engineer, may be used, but in no case less than three-fourths of that already specified; the live load assumed shall be proportional to the loading specified, with the same wheel spacing.

For girders and trusses of spans carrying more than one track, live loads shall be assumed as follows:

For two tracks, full live load.

For three or more tracks, full live load on the two tracks nearest the girder or truss, and 90% of full live load on all other tracks.

**Impact.**

204. The stresses due to the dynamic effect of the live load shall be added to the maximum computed live load stresses and shall be determined by the formula,

$$I = S \frac{400 - \frac{l}{2}}{400 + l}$$

in which,

$S$ = maximum computed live load stress for a single track.
$I$ = impact stress, dynamic effect of the live load.
$l$ = length, in feet, of the span which is loaded to produce the live load stress.

For girders and trusses of double-track bridges, the impact shall be taken from the full live load on one track only. The impact shall be only that from the live load on the track nearest the girder or truss.

For bridges with three or more tracks the impact shall be computed from the load on the track which, when loaded, produces the greatest live load stress in the member.

For bridges designed exclusively for electric traction, the impact shall be taken as one-third of that given by the impact formula.

**Centrifugal Force.**

205. On curves the centrifugal force shall be taken as a percentage of the live load, including impact, and assumed to act 6 ft. above the top of the rail. The percentages to be applied shall be as given in Table 1.

TABLE 1.

| Degree of curve | 0° 20′ | 0° 40′ | 1° | 2° | 3° | 4° | 5° | 6° | 7° | 8° | 9° | 10° | 11° | 12° |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Speed, in miles per hour | 72 | 72 | 72 | 59 | 53 | 46 | 41 | 38 | 35 | 33 | 31 | 29 | 28 | 27 |
| Percentage for centrifugal force. | 2 | 4 | 6 | 8 | 10 | 10 | 10 | 10 | 10 | 10 | 10 | 10 | 10 | 10 |

**Lateral Forces.**

206. Wind on Loaded Bridge.

The wind shall be considered as a moving load acting in any horizontal direction. The wind force on the bridge shall be taken at 30 lb. per sq. ft. of:

(*a*) One and one-half times the floor system as seen in elevation;
(*b*) The projection of all trusses;

but not less than 200 lb. per lin. ft. of loaded chord and 150 lb. per lin. ft. of unloaded chord.

The wind force on the train shall be taken as 300 lb. per lin. ft. on one track, applied 8 ft. above the top of the rail.

207. Wind on Unloaded Bridge.

If a wind force of 50 lb. per sq. ft. of surface as defined in Article 206, on the unloaded bridge, would produce greater stresses than the combined wind forces specified, those stresses shall be used in the members where such greater stresses occur.

208. Stability of Spans and Towers.

In calculating stability, spans and towers shall be considered as loaded on the leeward track with empty cars weighing 1 200 lb. per lin. ft., and subjected to a wind force of 300 lb. per lin. ft., applied 8 ft. above the top of the rail.

209. Sway of Engines.

The lateral force to provide for the effect of the sway of the engines, in addition to the wind loads specified, shall be a moving concentrated load of 20 000 lb., which may be applied at the base of rail in either horizontal direction at any point of the span.

**Longitudinal Force.**

210. The longitudinal force resulting from the starting and stopping of trains shall be 20% of the load on the engine drivers, and 5% of the load on the remainder of the train. This force shall be taken on one track only, and shall be assumed to act 6 ft. above the top of the rail.

In bridges where, by reason of continuity of members or frictional resistance, the longitudinal force will be directed largely to the abutments (such as ballasted deck bridges of only three or four spans), the longitudinal force shall be taken as one-half that specified.

**Reversal of Stress.**

211. Members subject to reversal of stress under the passage of the live load shall be proportioned for the stress requiring the larger section.

The designing stress shall be determined by obtaining the resultant stress of each kind, tension and compression, and adding 50% of the smaller stress to each.

**Combined Stresses.**

212. Members subject to both axial and bending stresses (including bending of posts due to floor-beam deflection) shall be so proportioned that the combined fiber stresses will not exceed the allowed axial stress. In members continuous over panel points, only three-fourths of the bending stress computed as for simple beams shall be added to the axial stress.

213. Members subject to stresses produced by a combination of dead load, live load, impact, and centrifugal force, with either lateral or longitudinal forces, or with bending due to lateral forces, may be proportioned for unit stresses 25% greater than those specified in Article 301; but the section of the member shall not be less than that required for the combined dead load, live load, impact, and centrifugal force.

**Secondary Stresses.**

214. Secondary stresses shall be avoided as far as practicable. In trusses without sub-paneling, secondary stresses due to distortion need not be considered in any member the width of which, measured parallel to the plane of flexure, is less than one-tenth of its length. Other secondary stresses shall be considered.

## SECTION 3.—UNIT STRESSES

**Unit Stresses.**

300. The dead load shall be proportioned for unit stresses 50% greater than those allowed in Article 301. (For convenience of calculations the live load

may be increased 50% beyond that specified in Article 203, and the resulting unit stresses, from dead and live loads combined, shall not exceed by more than 50% those given in Article 301.)

301. The allowable unit stresses to be used in proportioning the parts of a bridge shall be as follows:

| | For live load and all other forces, excepting dead load, in kips per square inch.* |
|---|---|
| (*a*) Structural and Rivet Steel: | |
| Axial tension, net section | 16.0 |
| Tension in extreme fibers of rolled shapes, built sections, and girders, net section | 16.0 |
| Axial compression, gross section | $\dfrac{16.0}{1+\dfrac{l^2}{13\,500r^2}}$ |
| but not to exceed the value for $\dfrac{l}{r} = 40$ | |
| in which, $l$ = length of the member, in inches; and, $r$ = least radius of gyration of the member, in inches. | |
| Compression flanges of plate girders | $\dfrac{16.0}{1+\dfrac{l^2}{2\,000b^2}}$ |
| in which, $l$ = the length, in inches, of the unsupported flange between lateral connections or knee-braces; and, $b$ = the flange width, in inches. | |
| Tension in extreme fibers of pins | 24.0 |
| Shear in plate girder webs, gross section | 10.0 |
| Shear in power-driven rivets and pins, outstanding legs of stiffener angles, and other steel parts in contact (Rivets driven and bucked by pneumatically or electrically operated hammers are considered power-driven.) | 24.0 |
| Bearing on turned bolts and hand-driven rivets | 20.0 |
| Bearing on expansion rollers, per linear inch | $0.6d$ |
| in which, $d$ = diameter of rollers, in inches. | |
| For cast-steel shoes and pedestals, the allowable unit stresses in structural steel shall apply. | |
| (*b*) High-Tension Steel: | |
| For members composed of steel of greater strength than structural grade the allowable stresses may be increased in proportion to the specified minimum yield point of the stronger steel, provided such yield point is not more than 70% of the ultimate strength. In the compression formulas the fractional term of the denominator shall be increased in the same proportion. | |

*1 kip equals 1 000 lb.

(*c*) Wooden Cross-Ties:

| | |
|---|---|
| Tension in extreme fibers: | |
| White oak and dense yellow pine | 1.8 |
| Dense Douglas fir | 1.2 |
| White pine, ordinary yellow pine, and spruce | 1.0 |

The maximum wheel load with 100% impact shall be considered as distributed over three ties.

(*d*) Masonry:

| | |
|---|---|
| Bearing pressure: | |
| Granite | 0.8 |
| Sandstone and limestone | 0.4 |
| Concrete | 0.6 |

**Provision for Overload.**

302. Members shall be so proportioned that an increase of the live load by 50% will not produce unit stresses in the members greater than those specified by Article 300 for dead load.

**Effective Bearing Area.**

303. The effective bearing area of pins, bolts, and rivets shall be the diameter multiplied by the length in bearing; except that for countersunk rivets, one-half the depth of the countersink shall be omitted.

**Effective Diameter of Rivets.**

304. In proportioning rivets, the nominal diameter of the rivet shall be used.

**Slenderness Ratio.**

305. The slenderness ratio (ratio of length to least radius of gyration) shall not exceed:

100 for main compression members.
120 for wind and sway bracing.
140 for single lacing.
200 for double lacing.
200 for riveted tension members.

## SECTION 4.—DETAILS OF DESIGN

**Thickness of Material.**

401. The minimum allowable thickness of material shall be $\frac{1}{4}$ in., except for fillers.

Gussets shall not be less than $\frac{3}{8}$ in. thick.

Metal subject to marked corrosive influences shall be thicker than otherwise would be required, or else protective measures shall be provided.

**Open Details.**

402. Details shall be so arranged as to give free access for inspection and painting. Water-pockets shall be avoided.

**Eccentric Connections.**

403. Eccentric connections shall be avoided if practicable, but, if they are unavoidable, the members shall be so proportioned that the combined fiber stresses will not exceed the allowed axial stress. Members shall be so arranged that their axes intersect in a point.

**Compression Members.**

404. Compression members shall be so designed that the metal will be concentrated, so far as practicable, in the webs and flanges, and so that the center of gravity of the section will be near the center line of the member.

The thickness of each web shall be not less than one-thirtieth of the distance between the lines of rivets connecting it to the flanges. The thickness of the cover-plates shall be not less than one-fortieth of the greatest distance between adjacent rivet lines.

**Outstanding Legs of Angles.**

405. The width of the outstanding legs of angles in compression, except those reinforced by plates, shall not exceed the following:

(*a*) For stringer flange angles, ten times the thickness.
(*b*) For main members carrying axial stress, twelve times the thickness.
(*c*) For bracing and other secondary members, fourteen times the thickness.

**Rigid Members.**

406. The two end panels of the bottom chords at each end of single-track pin-connected bridges, and the hip verticals and members with similar duties in all bridges, shall be rigid.

**Strength of Connections.**

407. Connections, excepting for transverse and sway-bracing members, shall have a strength at least equal to that of the members connected, based on the allowable unit stress in the member.

**Net Section.**

408. The net section of tension members shall be obtained as follows:

Compute the least sections, (*a*), (*b*), and (*c*):

(*a*) A zigzag section with all holes in the zigzag section deducted and the net diagonal distances taken at 90% of their value.
(*b*) A right section with the same number of holes deducted as appear in the zigzag section.
(*c*) A right section with the holes in that section only deducted.

If Section (*a*) is less than Section (*b*), Section (*b*) shall be used.

If Section (*a*) is equal to or greater than Section (*b*), but less than Section (*c*), Section (*a*) shall be used.

If Section (*a*) is equal to or greater than Section (*c*), Section (*c*) shall be used.

Rivet holes shall be taken as $\frac{1}{8}$ in. larger than the nominal diameter of the rivet.

**Effective Sections of Angles.**

409. If angles in tension are connected on one side of gusset-plates, the effective area will be taken as the net area of the connected leg plus one-half the area of the unconnected leg.

If angles in tension are connected so that bending cannot occur in any direction, the effective area shall be taken as the net area of the angle.

**Section at Pin-Holes.**

410. In pin-connected, riveted, tension members the net section beyond the pin-hole, parallel to the axis of the member, shall not be less than the net section of the member. The net transverse section through the pin-hole shall be at least one-third larger than the net section of the member. Riveted tension members shall be stitch-riveted where necessary, to make a compact member.

**Maximum Length of Rivets.**

411. If the grip of rivets carrying calculated stress exceeds four and one-half times the diameter, the number of rivets shall be increased at least 1% for each additional $\frac{1}{8}$ in. of grip. If the grip exceeds six times the diameter of the rivet, specially designed rivets shall be used.

**Spacing of Rivets.**

412. Rivets shall be proportioned by their nominal diameter. They shall be spaced not less than three diameters apart, center to center.

In the direction of stress they shall be spaced not farther apart than sixteen times the thickness of the thinnest plate connected; and at right angles to the direction of stress, not farther apart than thirty times that thickness,

except in the cover-plates of compression members, where the spacing may be forty times the thickness of the thinnest plate.

When two or more plates are in contact and riveted for compactness, the stitch rivets shall not be farther apart in any direction than twenty-four times the thickness of the thinnest outside plate connected. At the ends of built compression members the pitch of rivets in the direction of stress shall not exceed four times the diameter for a distance one and one-half times the width of the member.

**Edge Distance of Rivets.**

413. The distance from the center of a rivet to a sheared edge shall not be less than one and three-quarter times the diameter, nor to a rolled or planed edge less than one and one-half times the diameter, except in flanges of beams and channels, where the minimum distance may be one and one-quarter times the diameter.

The distance from the edge of a plate shall not exceed eight times the thickness of the plate.

**Compression Splices.**

414. Members subject to compression only, if faced for bearing, shall be spliced on four sides sufficiently to hold the abutting parts true to place and to transmit at least 25% of the stress through the splice material. The splice shall be as near the panel point as practicable. Members not faced for bearing shall be fully spliced.

**Extra Rivets Through Fillers.**

415. If splice plates are not in direct contact with the parts which they connect, rivets shall be used on each side of the joint in excess of the number required for direct contact to the extent of two extra lines for each intervening plate.

If rivets carrying stress pass through fillers, the fillers shall be extended beyond the connected member and the extension secured by enough additional rivets to carry one-half the stress passing through the fillers.

**Stay-Plates.**

416. In compression members the open sides shall be provided with lattice-bars and shall have stay-plates as near each end as practicable. Stay-plates shall be provided at intermediate points where the latticing is interrupted. In main members the length of the end stay-plates shall be not less than one and one-quarter times the distance between the lines of rivets connecting them to the outer flanges, and the length of intermediate stay-plates shall be not less than three-quarters of that distance. Their thickness shall be not less than one-fiftieth of the same distance.

417. In tension members composed of shapes the separate segments shall be connected by stay-plates, or by stay-plates and lattice-bars.

**Latticing.**

418. Lattice-bars of compression members shall be so spaced that the $\frac{l}{r}$ of the portion of the flange included between the lattice-bar connections will not be greater than two-thirds of the $\frac{l}{r}$ of the member, and in no case more than 40.

419. The latticing of compression members shall be proportioned to resist a shearing stress normal to the member, not less than that given by the following formulas:

$$R = \frac{4\,I}{C\,l}\left(20\,000 - p\right) \quad \ldots\ldots (1)$$

$$R = \frac{0.4p\,I}{C\,l} \quad \ldots\ldots (2)$$

in which,

$R$ = normal shearing stress, in pounds.
$I$ = moment of inertia of the section about an axis perpendicular to the plane of the latticing.
$C$ = distance from the neutral axis to the extreme fiber, in inches.
$l$ = length of the member, in inches.
$p$ = average compressive unit stress in the member = $\frac{P}{A}$.

The greater of the values given by these two formulas shall be used.

420. In compression members with cover-plates, the cover-plates shall be assumed to take one-half the shear.

421. The diameter of the rivet shall not exceed one-third of the width of the lattice-bar. Lattice-bars connecting to flanges more than 5 in. wide shall have at least two rivets in each end.

422. The angle between the lattice-bars and the axis of the member shall be not less than 45° for double latticing, and 60° for single latticing.

The thickness of single lattice-bars shall be not less than one-fortieth, and of double lattice-bars, not less than one-sixtieth, of the distance between rivets connecting them to the member.

Double latticing shall be riveted at intersections.

**Reinforcing Plates at Pin-Holes.**

423. Where necessary to give the required section or bearing area, the section at pin-holes shall be reinforced on each segment by plates. One plate on each side shall be as wide as the outstanding flanges will permit. These plates shall contain enough rivets, and be so connected, to transmit and distribute the bearing pressure uniformly over the full cross-section and to reduce the eccentricity of the segment to a minimum. At least one full-width plate on each segment shall extend to the far edge of the stay-plate and the others not less than 6 in. beyond the near edge.

**Forked Ends.**

424. Forked ends of compression members shall be avoided if practicable. Where forked ends are used, enough pin-plates shall be provided to give each jaw the full strength of the compression member. At least one of these plates shall extend to the farthest edge of the stay-plates, and the others not less than 6 in. beyond the near edge of the farther stay-plate.

**Floor-Beam Connections.**

425. Floor-beams preferably shall be square to the girders or trusses, and shall be riveted directly to the girders or to the posts of the trusses.

**End-Connection Angles.**

426. Stringers in through spans shall be riveted between the floor-beams. The connection angles shall be not less than 4 in. wide and ½ in. finished thickness. Solid floors shall be connected to the girders or trusses by angles not less than ½ in. finished thickness.

**Plate Girders.**

427. Design.

Plate girders shall be proportioned by the moment of inertia of their net sections, including the compression sides; or by assuming that the flanges are concentrated at their centers of gravity, but not beyond the backs of the flange angles. In the latter case, one-eighth of the gross section of the web, if properly spliced, may be considered as flange section. For girders having unusual forms of section, the moment of inertia method shall be used.

The gross section of the compression flange of a plate girder or a rolled beam shall not be less than the gross section of the tension flange.

428. Flange Plates.

Flanges of plate girders preferably shall be made without cover-plates or side plates unless angles of greater section than 6 by 6 in. by $\frac{7}{8}$ in. would otherwise be required.

Flange plates shall be equal in thickness, or shall diminish in thickness from the flange angles outward. No plate shall have a thickness greater than that of the flange angles.

When flange plates are used, at least one plate on each flange shall extend the full length of the girder, and on through bridges, an end and corner cover-plate shall be used. Any additional flange plates shall be of such length as to allow two rows of rivets of the regular pitch to be placed at each end of the plate, beyond the theoretical point required, and there shall be a sufficient number of rivets at the ends of each plate to transmit its stress before the theoretical point of the next outside plate is reached.

429. Flange Splices.

Flange members that are spliced shall be covered by extra material equal in section to the member spliced. There shall be enough rivets on each side of the splice to transmit the stress of the parts cut.

Flange angles shall be spliced with angles.

No two members shall be spliced at the same flange cross-section.

430. Web Splices.

Splices in the webs of plate girders shall be designed for both shear and bending.

431. Flange Rivets.

The flanges of plate girders shall be connected to the web with enough rivets to transfer to the flange section the horizontal shear at any point, combined with any load that is applied directly on the flange. Where ties rest on the flange, one wheel load shall be assumed to be distributed over three ties.

432. Stiffeners.

Stiffener angles shall be placed at end bearings and at points of concentrated load. Such stiffeners shall not be crimped. The outstanding legs shall be proportioned for bearing and shall extend out as nearly as practicable to the edge of the flange angles.

433. Intermediate Stiffeners.

Webs of plate girders shall be stiffened by pairs of angles approximately at distances given by the formula,

$$d = 100\, t \sqrt{\frac{16\,000}{s} - 1}$$

in which,

$d$ = the clear distance, in inches, between the stiffeners.

$t$ = the thickness, in inches, of the web.

$s$ = the equivalent shearing stress on the gross vertical section, in pounds per square inch, obtained by dividing the sum of the live load shear, impact shear, and two-thirds of the dead load shear, by the gross area of the web.

but the distance shall not exceed (*a*) 6 ft., or (*b*) the depth between the flanges.

If the depth between the flanges is less than fifty times the thickness of the web, intermediate stiffeners may be omitted.

The thickness of intermediate stiffeners shall be not less than one-sixteenth of the width of the outstanding legs. The outstanding leg shall reach as near to the edge of the flange angle as practicable. The widths shall not be less than 2 in. plus one-thirtieth of the depth of the web between flanges.

434. Brackets.

The top flanges of through plate girders shall be rigidly braced laterally, at panel points of the bridge, by brackets with web-plates. The brackets shall extend to the top of the main girders and shall be as large as the clearance will allow. They shall be attached rigidly to the web-plate of the girder and to the top flange of the floor-beam. In solid floors these braces shall be not more than 12 ft. apart.

**Lateral Bracing.**

435. Bottom lateral bracing shall be provided in all spans except deck plate girder spans less than 50 ft. long. Continuous steel or concrete floors shall be considered as lateral bracing.

Top lateral bracing shall be provided in deck spans, and in through spans having sufficient head-room.

**Portal and Sway-Bracing.**

436. Through truss spans shall have portal bracing, with knee-braces, as deep as the specified clearance will allow.

Through truss spans shall have sway-bracing at every intermediate panel point if the height of the trusses is enough to allow a depth of 6 ft. or more for the bracing. When the height of the trusses will not allow that depth, the top lateral struts shall be of the same depth as the chord, and shall have knee-braces as large as the clearance will allow.

Deck truss spans shall have sway-bracing at every panel point. The top lateral loads shall be carried to the supports by means of a complete top lateral system in the planes of the top chords and the end posts.

**Rigid Bracing.**

437. Lateral bracing shall be rigid, with not less than three rivets in each end connection.

When a double system of bracing is used, both systems may be considered effective simultaneously, if the members meet the requirements as both tension and compression members.

**Cross-Frames.**

438. In deck plate girder spans there shall be cross-frames at both ends proportioned to resist centrifugal and lateral forces, and intermediate cross-frames at intervals not exceeding 18 ft.

**Viaduct Towers.**

439. Viaduct bents preferably shall be composed of two supporting columns, and towers shall be formed by uniting these bents in pairs.

The towers shall be braced both transversely and longitudinally with a double system of rigid diagonals, and shall have longitudinal and transverse struts at caps and bases and at all intermediate panel points.

In double-track towers, the bracing shall be designed to transmit the longitudinal force to both sides.

Where long spans are supported on short single bents, such bents shall have hinged ends, or else the columns and anchorage shall be proportioned for the bending stresses produced by temperature changes.

The bottom struts of viaduct towers shall be proportioned for the calculated stresses, or for stresses in tension or compression not less than one-fourth the dead load reaction on one pedestal. The column bearings shall be designed to permit expansion and contraction of the tower bracing.

The columns preferably shall have a batter transversely of 1 horizontal to 6 vertical for single-track viaducts, and 1 horizontal to 8 vertical for double-track viaducts.

**Eye-Bars.**

440. The thickness of eye-bars shall be not less than 1 in., nor greater than 2 in. The thickness plus $\frac{1}{4}$ in. shall not be less than one-eighth of the width.

The section of the head through the center of the pin-hole shall exceed that of the body of the bar by at least 37½ per cent. The diameter of the pin shall be not less than three-fourths of the width of the bar.

The form of the head shall be submitted to the Engineer for approval before the bars are made.

**Eye-Bar Packing.**

441. The eye-bars of a set shall be symmetrical about the plane of the truss and as nearly parallel as practicable. The inclination of any bar to the plane of the truss shall not exceed ⅛ in. to the foot. The bars shall be packed close, held against lateral movement, and so arranged that those in the same panel will not be in contact.

**Expansion.**

442. Provision shall be made for expansion and contraction of spans at the rate of 1 in. in 100 ft. In spans more than 400 ft. long, provision shall be made for expansion in the floor.

**End Bearings.**

443. In spans more than 70 ft. long there shall be rollers at one end. Shorter spans shall be arranged to slide on smooth surfaces.

Bearings and ends of spans shall be secured against lateral movement.

**Rollers.**

444. Expansion rollers shall be not less than 6 in. in diameter. They shall be coupled together with substantial side-bars and geared to the upper and lower plates. The parts shall be so arranged that the rollers can be cleaned readily.

**Pedestals and Shoes.**

445. Pedestals and shoes preferably shall be made of cast steel. The difference in width between the top and bottom bearing surfaces shall not exceed twice the distance between them. For hinged bearings, the distance shall be measured from the center of the pin. In built pedestals and shoes, the web-plates and the angles connecting them to the base plate shall be not less than ¾ in. thick. If the size of the pedestal permits, the webs shall be rigidly connected transversely. The minimum thickness of the metal in cast-steel pedestals shall be 1 in. Pedestals and shoes shall be so constructed that the load will be distributed uniformly over the entire bearing surface. Spans more than 70 ft. long shall have hinged bearings at both ends.

**Inclined Bearings.**

446. For spans on an inclined grade and without hinged bearings, the sole or masonry plates shall be beveled so that the masonry surfaces will be level.

**Anchorage for Towers.**

447. Anchor-bolts for viaduct towers and similar structures shall be designed to engage a mass of masonry the weight of which is at least one and one-half times the uplift.

Anchor-bolts for trusses and girders shall not be less than 1¼ in. in diameter and extend into the masonry not less than 12 in.

**Camber.**

448. The length of truss members shall be such that the camber will be equal to the deflection produced by the dead load plus full train load without impact; ordinarily this will be effected by increasing the length of the top chord ⅛ in. for each 10 ft.

## SECTION 5.—WORKMANSHIP

**General.**

501. The workmanship and finish shall be equal to the best general practice in modern bridge shops. Material at the shops shall be kept clean and protected from the weather as far as practicable.

**Straightening.**

502. Material shall be straight before being laid off or worked. Straightening shall be done by methods that are not injurious. Sharp kinks or bends shall be cause for rejection.

**Shearing.**

503. Shearing and chipping shall be done neatly and accurately. If carrying calculated stress, structural steel exceeding $\frac{3}{4}$ in. in thickness, and alloy steel exceeding $\frac{1}{2}$ in. in thickness, shall have $\frac{1}{4}$ in. of metal planed from the sheared edges. Re-entrant angles shall be filleted before cutting.

**Punching and Reaming.**

504. Material, forming parts of a member composed of not more than five thicknesses of metal, may be punched $\frac{1}{16}$ in. larger than the nominal diameter of the rivets, whenever the thickness (of the metal) is not greater than $\frac{3}{4}$ in. for structural steel, or $\frac{1}{2}$ in. for alloy steel. When there are more than five thicknesses, or when any of the main material is thicker than $\frac{3}{4}$ in. in structural steel, or $\frac{1}{2}$ in. in alloy steel, all the holes shall be punched $\frac{1}{16}$ in. smaller, and after assembling, reamed $\frac{1}{16}$ in. larger, than the nominal diameter of the rivets; except that when the metal is thicker than the diameter of the rivet minus $\frac{1}{8}$ in., the holes shall be drilled.

505. The diameter of the die shall not exceed the diameter of the punch by more than $\frac{1}{32}$ in. If any holes must be enlarged to admit the rivets, they shall be reamed. Holes must be clean cut, without torn or ragged edges. Poor matching of holes may be cause for rejection.

**Reaming After Assembling.**

506. Reaming shall be done after the pieces forming a built member are assembled and so firmly bolted together that the surfaces are in close contact. Burrs on the outside surface shall be removed. The pieces shall be taken apart before riveting, if necessary, and any shavings removed. When it is necessary to take the members apart for shipping or handling, the pieces reamed together shall be so marked that they may be re-assembled in the same position in the final setting up. No interchange of reamed parts will be permitted.

**Reamed Holes.**

507. Reamed holes shall be cylindrical, perpendicular to the member, and not more than $\frac{1}{16}$ in. larger than the nominal diameter of the rivets. Reamers preferably shall be directed by mechanical means. Outside burrs shall be removed.

**Drilled Holes.**

508. Drilled holes shall be $\frac{1}{16}$ in. larger than the nominal diameter of the rivet. Burrs on the outside surfaces shall be removed. Drilling shall be accurately done. Poor matching of holes may be cause for rejection.

**Reaming and Drilling.**

509. Reaming and drilling shall be done with twist drills.

**Shop Assembling.**

510. The parts of riveted members shall be well pinned and firmly drawn together with bolts before riveting is commenced. The drifting done during assembling shall be only such as to bring the parts into position and not sufficient to enlarge the holes or distort the metal.

**Field Connections.**

511. Solid floor sections shall be assembled to the girders or trusses, or to suitable frames, in the shop, and the end connections made to fit.

512. Riveted trusses shall be assembled in the shop to line and fit and the holes for field connections drilled or reamed while so assembled; or these connections may be reamed to proper metal templates so placed as to result

in fair holes for the connections. Holes for other field connections, except those in lateral, longitudinal, and sway-bracing, shall be reamed or drilled to a metal template.

**Match-Marking.**

513. Connecting parts assembled in the shop for the purpose of reaming or drilling holes in field connections shall be match-marked, and a diagram showing such marks shall be furnished the Engineer.

**Rivets.**

514. The size of rivets called for on the plans shall be the size before heating.

515. Rivet heads shall be of approved shape and of uniform size for the same diameter of rivet. They shall be full, neatly made, concentric with the rivet holes, and in full contact with the surface of the member.

**Riveting.**

516. Rivets shall be heated uniformly to a light cherry red and driven while hot. They shall be free from slag, scale, and carbon deposit. When driven, they shall completely fill the holes. Loose, burned, or otherwise defective rivets shall be replaced. In removing rivets, care shall be taken not to injure the adjacent metal and, if necessary, they shall be drilled out. Caulking or re-cupping shall not be done.

517. Rivets shall be driven by direct-acting riveters where practicable. The pressure shall be continued after the upsetting has been completed.

518. When rivets are driven with a pneumatic riveting hammer, a pneumatic bucker shall be used for holding up, when practicable.

**Field Rivets.**

519. Field rivets shall be furnished in excess of the nominal number required to the amount of 15%, plus ten rivets, for each size and length.

520. Field rivets shall be free from fins on the under side of the head.

**Turned Bolts.**

521. Where turned bolts are used to transmit shear, the holes shall be reamed parallel and the bolts shall make a tight fit with the threads entirely outside the holes. A washer not less than $\frac{1}{4}$ in. thick shall be used under each nut.

**Lacing-Bars.**

522. The ends of lacing-bars with single rivets shall be neatly rounded, unless otherwise specified.

**Fit of Stiffeners.**

523. Stiffeners under the top flanges of deck girders and at all bearing points shall be milled or ground to bear against the flange angles. Other stiffeners must fit sufficiently tight against the flange angles to exclude water after being painted. Fillers and splice-plates shall fit within $\frac{1}{4}$ in. at each end.

**Web-Plates.**

524. Web-plates of girders which have no cover-plates may be $\frac{1}{8}$ in. above or below the backs of the top flange angles. Web-plates of girders which have cover-plates may be $\frac{1}{2}$ in. less in width than the distance back to back of flange angles.

525. If web-plates are spliced, there shall be not more than $\frac{3}{8}$-in. clearance between ends of plates.

**Facing Floor-Beams, Stringers, and Girders.**

526. Floor-beams, stringers, and girders having end connection angles shall be made of exact length. If facing is necessary, the thickness of the end connection angles shall not be reduced more than $\frac{1}{8}$ in. at any point.

**Finished Members.**

527. Finished members shall be true to line and free from twists, bends, and open joints.

**Abutting Joints.**

528. Joints in compression members and girder flanges, and where so specified on the drawings, in tension members, shall have the abutting surfaces faced and brought to an even bearing. Where joints are not faced, the opening shall not exceed $\frac{1}{4}$ in.

**Eye-Bars.**

529. Eye-bars shall be straight, true to size, and free from twists, folds in the neck or head, and other defects. The heads shall be made by upsetting, rolling, or forging, and not by welding. The form of the head will be determined by the dies in use at the works where the eye-bars are made, if they are satisfactory to the Engineer. The thickness of the head and neck shall not over-run more than $\frac{1}{16}$ in.

530. Eye-bars that are to be placed side by side in the structure shall be bored so accurately that upon being placed together, pins $\frac{1}{32}$ in. less in diameter than the pin-holes will pass through the holes at both ends at the same time without driving.

**Annealing.**

531. Before boring, all eye-bars shall be annealed or heat-treated to produce the required physical qualities, and shall be straightened.

532. Other steel which has been partially heated shall be annealed unless it is to be used in minor parts.

**Boring Pin-Holes.**

533. Pin-holes shall be bored true to gauge, smooth, straight, at right angles with the axis of the member, and parallel with each other, unless otherwise required. The variation from the specified distance from outside to outside of pin-holes in tension members, or from inside to inside of pin-holes in compression members, shall not exceed $\frac{1}{32}$ in. In built-up members the boring shall be done after the member is riveted.

534. The diameter of the pin-hole shall not exceed that of the pin by more than $\frac{1}{50}$ in. for pins 8 in. or less in diameter, nor $\frac{1}{32}$ in. for larger pins.

**Pins and Rollers.**

535. Pins more than 7 in. in diameter shall be forged and annealed.

Pins and rollers shall be turned accurately to gauge.

They shall be straight, smooth, and free from flaws.

For pins 9 in. or more in diameter there shall be a 2-in. hole bored longitudinally through the center.

**Upset Ends.**

536. Bars with screw ends shall be upset so that the section at the root of the thread will be at least 15% greater than in the body of the bar.

**Screw-Threads.**

537. Screw-threads shall fit the nut-threads. They shall be U. S. Standard, except that for pin ends of diameters greater than $1\frac{3}{8}$ in., they shall be made with six threads to an inch.

**Bearing Surfaces Planed.**

538. The top and bottom surfaces of base-plates and cap-plates of columns and pedestals, except surfaces to be in contact with masonry, shall be planed or else hot-straightened. The parts of members in contact with them shall be faced to fit. Connection angles for base-plates and cap-plates shall be riveted to compression members before the members are faced.

539. Sole plates of plate girders shall have full contact with the girder flanges. Sole plates and masonry plates shall be planed or else hot-straightened.

Cast pedestals shall be planed on the surface in contact with steel. The bottom surfaces to rest on masonry shall be rough-finished.

**Pilot Nuts.**

540. Two pilot nuts and two driving nuts for each size of pin shall be furnished, unless otherwise specified.

**Shop Painting.**

541. Unless otherwise specified, steel work shall be thoroughly cleaned and given one coat of approved paint, after it has been accepted by the Inspector and before leaving the shop. The paint shall be applied in a workmanlike manner, and well worked into joints and open spaces. Cleaning shall be done with steel brushes, hammers, scrapers, and chisels, or by other equally effective means. Oil, paraffin, and grease shall be removed by the use of benzine or gasoline. Loose dirt shall be brushed off with a dry bristle brush. Surfaces not in contact but inaccessible after assembling shall be painted.

**Machined Surfaces.**

542. Machined-finished surfaces of steel, except abutting joints and baseplates, shall be coated with white lead and tallow applied hot as soon as the surfaces have been finished and accepted by the Inspector.

**Facilities for Inspection.**

543. The Contractor shall afford the Inspector, without charge, facilities for inspection of materials and workmanship in the shop. The Inspector shall be allowed free access to the necessary parts of the works.

**Material Orders and Shipping Statements.**

544. The Contractor shall furnish to the Engineer as many copies of material orders and shipping statements as the Engineer may direct. The weights of the individual members shall be shown on the statements.

**Notice of Beginning Work.**

545. The Contractor shall give ample notice to the Engineer of the beginning of rolling in the mill, and of work in the shop, so that inspection may be provided. No material shall be rolled, nor work done in shop, before the Engineer has been so notified.

**Inspector's Authority.**

546. The Inspector shall have authority to reject materials or workmanship which do not meet the requirements of these specifications. In case of dispute, the Contractor may appeal to the Engineer, whose decision shall be final.

**Rejection.**

547. The acceptance of any material or finished members by the Inspector shall not be a bar to their subsequent rejection, if found defective.

Rejected material and workmanship shall be replaced promptly or made good by the Contractor.

## SECTION 6.—FULL-SIZE TESTS OF EYE-BARS

**Eye-Bars.**

601. The acceptance of the eye-bars shall depend upon the results of full-size tests.

**Number of Full-Size Tests.**

602. The number and size of the bars to be tested shall be stipulated by the Engineer before the mill order has been placed. The number shall not exceed 5% of the whole number of bars ordered, with a minimum of two bars on small orders.

**Selection of Test Bars.**

603. The test bars shall be of the same section as the bars to be used in the structure and of the same length, if within the capacity of the testing

machine. They shall be selected by the Inspector from the finished bars. Test bars representing bars too long for the testing machine shall be selected from the full-length bar material after the heads on one end have been formed. Then they shall be cut and the second head formed, making a bar of the greatest length that can be tested.

604. The minimum requirements for full-size eye-bar tests shall be as follows:

| | |
|---|---|
| Yield point, in pounds per square inch | 33 000 |
| Ultimate strength, in pounds per square inch | 60 000 |
| Elongation in 18 ft., percentage | 12 |

The elongation shall be measured in the body of the bar, including the fracture. The fracture shall show a silky or finely granular structure throughout.

**Retests.**

605. If a bar fails to meet the requirements of Article 604 two additional bars of the same size and from the same mill heat shall be tested. The bars represented by the test may be re-annealed before the additional bars are tested. If two of the three bars tested fail, the bars of that test and mill heat shall be rejected.

606. A record of the annealing charges showing the bars in each charge and details of the treatment as to temperature and time, shall be furnished to the Engineer.

**Payment for Test Bars.**

607. Bars tested full-size shall be paid for by the Company at the same rate as the bars accepted, if they meet the requirements of the specifications. Bars which fail to meet these requirements, and bars rejected as a result of tests, shall not be paid for by the Company.

## SECTION 7.—WEIGHING AND SHIPPING

**Pay Weight.**

701. The payment in pound-price contracts shall be based on the scale weight of the metal in the fabricated structure, including field rivets shipped. The weight of the field paint and cement, if furnished, boxes and barrels used for packing, and material used for staying or supporting members on cars, shall not be included.

Any weight in excess of 1½% more than the computed weight shall not be included.

**Variance in Weight.**

702. If the scale weight of any member is less than 97½% of the computed weight, it shall be cause for rejection.

**Weight of Steel and Iron.**

703. The weight of steel shall be assumed as 0.2833 lb. per cu. in. The weight of cast iron shall be assumed as 0.2604 lb. per cu. in.

**Computed Weight.**

704. The weights of rolled shapes, and of plates up to and including 36 in. in width, shall be computed on the basis of their nominal weights and dimensions, as shown on the approved shop drawings, deducting for copes, cuts, and open holes. For plates more than 36 in. in width, there shall be added to the nominal weight one-half the allowed percentage of over-run in weight given in Articles 827 and 828.

The weight of the heads of shop-driven rivets shall be included in the computed weight.

The weights of castings shall be computed from the dimensions shown on the approved shop drawings, with 10% added for fillets and over-run.

**Weighing.**

705. The finished work shall be weighed in the presence of the Inspector, if practicable. The Contractor shall furnish satisfactory scales and do the handling of the material for weighing.

**Marking and Shipping.**

706. The weight shall be marked on members weighing more than 5 tons. Bolts and rivets of one length and diameter, and loose rivets and washers of each size, shall be packed separately. Pins, other small parts, and small packages of bolts, rivets, washers, and nuts, shall be shipped in boxes, crates, kegs, or barrels, but the gross weight of any package shall not exceed 300 lb. On the outside of each package shall be marked plainly a list and description of the material contained therein.

**Loading Long Girders.**

707. Long girders shall be so loaded and marked that they will arrive at the bridge site in position for erection without turning.

**Shipment of Anchorage Material.**

708. Anchor-bolts, washers, and other anchorage or grillage materials shall be shipped in time to be built into the masonry.

## PART II.—MATERIALS

### SECTION 8.—STRUCTURAL STEEL*

**Process.**

801. The steel shall be made by the open-hearth process.

**Chemical Composition.**

802. The steel shall conform to the following requirements as to chemical composition:

| | | Structural steel. | Rivet steel. |
|---|---|---|---|
| Phosphorus | Acid | Not more than 0.06% | Not more than 0.04% |
| | Basic | " " " 0.04% | " " " 0.04% |
| Sulfur | | " " " 0.05% | " " " 0.045% |

**Ladle Analyses.**

803. An analysis of each melt of steel shall be made by the manufacturer to determine the percentages of carbon, manganese, phosphorus, and sulfur. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus determined shall be reported to the Engineer or his representative, and shall conform to the requirements specified in Article 802.

**Check Analyses.**

804. Analysis may be made by the Engineer from finished material representing each melt. The phosphorus and sulfur content thus determined shall not exceed that specified in Article 802 by more than 25 per cent.

**Tension Tests.**

805. The material shall conform to the requirements as to tensile properties given in Table 2.

806. In order to meet the required minimum tensile strength of full-size annealed eye-bars, the Engineer may determine the tensile strength to be obtained in specimen tests; the range shall not exceed 14 000 lb. per sq. in., and the maximum shall not exceed 74 000 lb. per sq. in. The material shall conform to the requirements as to physical properties other than tensile strength, specified in Articles 805, 808, 809, and 811.

* In substantial agreement with A. S. T. M. Specifications—Serial Designation A7-24.

807. The yield point shall be determined by the drop of the beam of the testing machine.

**Modification in Elongation.**

808. For structural steel more than ¾ in. in thickness, or diameter, a deduction of 0.25% from the percentage of elongation in 8 in. specified in Article 805 shall be made for each ⅛ in. of the specified thickness or diameter more than ¾ in.; but the reduced percentage shall not be less than 18 per cent.

809. For structural steel less than ⅝ in. in thickness, or diameter, a deduction of 1.25% from the percentages of elongation in 8 in. specified in Article 805 shall be made for each ⅛ in. in thickness or diameter less than ⅝ in.

TABLE 2.

| Properties considered. | Structural steel. | Rivet steel. |
|---|---|---|
| Tensile strength, in pounds per square inch | 55 000–65 000* | 46 000–56 000 |
| Yield point, minimum, pounds per square inch | 0.5 tensile strength | 0.5 tensile strength |
| but in no case less than | 30 000 | 25 000 |
| Elongation in 8 in., minimum, percentage | 1 500 000† / Tensile strength. | 1 500 000 / Tensile strength. |
| Elongation in 2 in., minimum, percentage | 22 | ..... |

* See Article 806.

† See Article 808.

**Bend Tests.**

810. Bend test specimens, except as specified in Articles 811 and 812, shall stand being bent cold through 180° without cracking on the outside of the bent portion, as follows: Material ¾ in. or less in thickness, flat on itself; material more than ¾ in., to and including 1¼ in., in thickness, around a pin the diameter of which is equal to the thickness of the specimen; and material more than 1¼ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

811. Bend test specimens for eye-bar flats shall stand being bent cold through 180° without cracking on the outside of the bent portion as follows: Material ¾ in. or less in thickness, around a pin the diameter of which is equal to the thickness of the specimen; material more than ¾ in., to and including 1¼ in., in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen; and material more than 1¼ in. in thickness, around a pin the diameter of which is equal to three times the thickness of the specimen.

812. Bend test specimens for rivet steel shall stand being bent cold through 180° flat on themselves without cracking on the outside of the bent portion.

**Test Specimens.**

813. Test specimens shall be prepared for testing from the material in its rolled or forged condition, except as specified in Articles 814 and 815.

814. Test specimens for annealed material shall be prepared from the material as annealed for use, or from a short length of a full section similarly treated.

815. Test specimens for rivet bars which have been cold-drawn shall be normalized before testing.

816. Test specimens shall be taken longitudinally and, except as specified in Articles 818, 819, and 820, shall be of the full thickness or section of material as rolled.

817. Test specimens for plates, shapes, and flats may be machined to the form and dimensions shown in Fig. 3, or with both edges parallel. Bend test specimens for eye-bar flats may have three rolled sides.

818. Tension test specimens for material more than 1½ in. in thickness or diameter, except pins and rollers, may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in., or they may conform to the dimensions shown in Fig. 4.

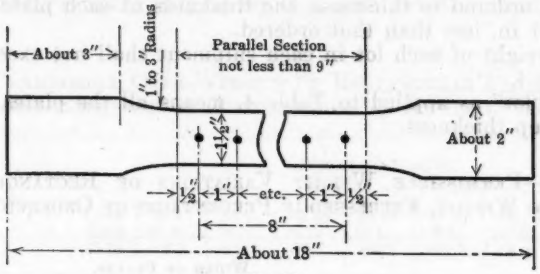
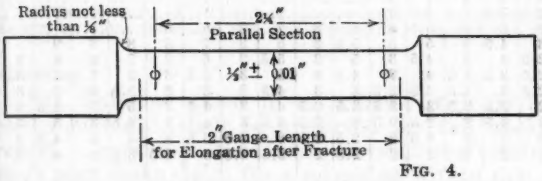



FIG. 3.

819. Bend test specimens for material more than 1½ in. in thickness or diameter, except eye-bar flats, pins, and rollers, may be machined to a thickness or diameter of at least ¾ in., or to 1 in. by ½ in. in section.

820. Tension test specimens for pins and rollers shall conform to the dimensions shown in Fig. 4, and bend test specimens shall be 1 in. by ½ in. in section.




Note:- The Gauge Length, Parallel Section and Fillets shall be as shown, but the Ends may be of any Shape to fit the Holders of the Testing Machine in such a way that the Load will be axial.

FIG. 4.

821. Test specimens for pins and rollers shall be taken so that the axis is 1 in. from the surface.

822. The machined sides of rectangular bend test specimens may have the corners rounded to a radius of not more than ⅛ in.

**Number of Tests.**

823. One tension and one bend test shall be made from each melt; except that if material from one melt differs ⅜ in. or more in thickness, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

824. If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

825. If the percentage of elongation of any tension test specimen is less than that specified in Article 805 and any part of the fracture is more than ¾ in. from the center of the gauge length of a 2-in. specimen, or is outside the middle-third of the gauge length of an 8-in. specimen, as indicated by scribe scratches marked on the specimen before testing, a re-test shall be allowed.

**Permissible Variations.**

826. The cross-section or weight of each piece of steel shall not vary more than 2.5% from that specified; except in the case of sheared plates, which shall be covered by the permissible variations specified in Articles 827 and 828. (One cubic inch of roller steel is assumed to weigh 0.2833 lb.)

827. When ordered to weight per square foot, the weight of each lot in each shipment shall not vary from the weight ordered more than the amount given in Table 3.

The term "lot" as applied to Table 3 means all the plates of each group width and group weight.

828. When ordered to thickness, the thickness of each plate shall not vary more than 0.01 in. less than that ordered.

The over-weight of each lot in each shipment shall not exceed the amount given in Table 4.

The term "lot" as applied to Table 4, means all the plates in each group width and group thickness.

TABLE 3.—PERMISSIBLE WEIGHT VARIATIONS OF RECTANGULAR PLATES ORDERED TO WEIGHT, EXPRESSED IN PERCENTAGES OF ORDERED WEIGHTS.*

| Ordered weight, in pounds per square foot. | Width of Plates. | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Less than 48 in. | | 48 to 60 in., excl. | | 60 to 72 in., excl. | | 72 to 84 in., excl. | | 84 to 96 in., excl. | | 96 to 108 in., excl. | | 108 to 120 in., excl. | | 120 to 132 in., excl. | | 132 in., or more. | |
| | Over. | Under. | Over. | Under. | Over. | Under. | Over. | Under. | Over. | Under. | Over. | Under. | Over. | Under. | Over. | Under. | Over. | Under. |
| Less than 5 | 5 | 3 | 5.5 | 3 | 6 | 3 | 7 | 3 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 5 to 7.5 excl. | 4.5 | 3 | 5 | 3 | 5.5 | 3 | 6 | 3 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 7.5 " 10 " | 4 | 3 | 4.5 | 3 | 5 | 3 | 5.5 | 3 | 6 | 3 | 7 | 3 | 8 | 3 | .... | .... | .... | .... |
| 10 " 12.5 " | 3.5 | 2.5 | 4 | 3 | 4.5 | 3 | 5 | 3 | 5.5 | 3 | 6 | 3 | 7 | 3 | 8 | 3 | 9 | 3 |
| 12.5 " 15 " | 3 | 2.5 | 3.5 | 2.5 | 4 | 3 | 4.5 | 3 | 5 | 3 | 5.5 | 3 | 6 | 3 | 7 | 3 | 8 | 3 |
| 15 " 17.5 " | 2.5 | 2.5 | 3 | 2.5 | 3.5 | 2.5 | 4 | 3 | 4.5 | 3 | 5 | 3 | 5.5 | 3 | 6 | 3 | 7 | 3 |
| 17.5 " 20 " | 2.5 | 2 | 2.5 | 2.5 | 3 | 2.5 | 3.5 | 2.5 | 4 | 3 | 4.5 | 3 | 5 | 3 | 5.5 | 3 | 6 | 3 |
| 20 " 25 " | 2 | 2 | 2.5 | 2 | 2.5 | 2.5 | 3 | 2.5 | 3.5 | 2.5 | 4 | 3 | 4.5 | 3 | 5 | 3 | 5.5 | 3 |
| 25 " 30 " | 2 | 2 | 2 | 2 | 2.5 | 2 | 2.5 | 2.5 | 3 | 2.5 | 3.5 | 3 | 4 | 3 | 4.5 | 3 | 5 | 3 |
| 30 " 40 " | 2 | 2 | 2 | 2 | 2 | 2 | 2.5 | 2 | 2.5 | 2.5 | 3 | 2.5 | 3.5 | 3 | 4 | 3 | 4.5 | 3 |
| 40 or more | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2.5 | 2 | 2.5 | 2.5 | 3 | 2.5 | 3.5 | 3 | 4 | 3 |

* The weight per square foot of individual plates shall not vary from the ordered weight by more than one and one-third times the amount given in this table.

**Finish.**

829. The finished material shall be free from injurious defects and shall have a workmanlike finish.

**Marking.**

830. The name or brand of the manufacturer and the melt number shall be legibly stamped or rolled on all finished material, except that when loaded for shipment, rivet and lattice-bars and other small sections shall be properly separated and marked for identification. The identification marks shall be legibly stamped on the end of each pin and roller. The melt number shall be legibly marked, by stamping if practicable, on each test specimen.

**Inspection.**

831. The Inspector representing the Engineer shall have free entry, at all times while work on the contract of the Engineer is being performed, to all parts of the Manufacturer's works which concern the manufacture of the material ordered. The Manufacturer shall afford the Inspector, without charge, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analyses) and inspection, shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

**Rejection.**

832. Unless otherwise specified, any rejection based on tests made in accordance with Article 804 shall be reported within five working days from the receipt of samples.

833. Material which shows injurious defects subsequent to its acceptance at the Manufacturer's works will be rejected, and the Manufacturer shall be notified.

TABLE 4.—PERMISSIBLE OVER-WEIGHTS OF RECTANGULAR PLATES ORDERED TO THICKNESS EXPRESSED IN PERCENTAGES OF NOMINAL WEIGHTS.*

| Ordered thickness, in inches. | Width of Plates. | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Less than 48 in. | 48 to 60 in., excl. | 60 to 72 in., excl. | 72 to 84 in., excl. | 84 to 96 in., excl. | 96 to 108 in., excl. | 108 to 120 in., excl. | 120 to 132 in., excl. | 132 in., or more. |
| Less than 1/8 | 9 | 10 | 12 | 14 | ..... | ..... | ..... | ..... | ..... |
| 1/8 to 3/16 excl | 8 | 9 | 10 | 12 | ..... | ..... | ..... | ..... | ..... |
| 3/16 " 1/4 " | 7 | 8 | 9 | 10 | 12 | ..... | ..... | ..... | ..... |
| 1/4 " 5/16 " | 6 | 7 | 8 | 9 | 10 | 12 | 14 | 16 | 19 |
| 5/16 " 3/8 " | 5 | 6 | 7 | 8 | 9 | 10 | 12 | 14 | 17 |
| 3/8 " 7/16 " | 4.5 | 5 | 6 | 7 | 8 | 9 | 10 | 12 | 15 |
| 7/16 " 1/2 " | 4 | 4.5 | 5 | 6 | 7 | 8 | 9 | 10 | 13 |
| 1/2 " 5/8 " | 3.5 | 4 | 4.5 | 5 | 6 | 7 | 8 | 9 | 11 |
| 5/8 " 3/4 " | 3 | 3.5 | 4 | 4.5 | 5 | 6 | 7 | 8 | 9 |
| 3/4 " 1 " | 2.5 | 3 | 3.5 | 4 | 4.5 | 5 | 6 | 7 | 8 |
| 1 or more | 2.5 | 2.5 | 3 | 3.5 | 4 | 4.5 | 5 | 6 | 7 |

* The weight of individual plates ordered to thickness shall not exceed the nominal weight by more than one and one-third times the amount given in this table.

**Rehearing.**

834. Samples tested in accordance with Article 804, which represent rejected material, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the Manufacturer may make claim for a rehearing within that time.

## SECTION 9.—SILICON STEEL

**Process.**

901. The steel shall be made by the open-hearth process.

**Discard.**

902. A sufficient discard shall be made from each ingot to secure freedom from injurious piping and undue segregation.

**Chemical Composition.**

903. The steel shall conform to the following requirements as to chemical composition:

| | Ladle Analysis. | Check Analysis. |
|---|---|---|
| Carbon, percentage | not more than 0.40 | not more than 0.44 |
| Phosphorus, percentage { Acid | " " " 0.06 | " " " 0.075 |
| Phosphorus, percentage { Basic | " " " 0.04 | " " " 0.05 |
| Sulfur, percentage | " " " 0.05 | " " " 0.063 |
| Silicon, percentage | " less than 0.20 | " less than 0.18 |

**Ladle Analyses.**

904. An analysis of each melt of steel shall be made by the Manufacturer to determine the percentages of carbon, manganese, phosphorus, sulfur, and silicon. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus determined shall be reported to the Engineer or his representative and shall conform to the requirements specified in Article 903 for ladle analysis.

**Check Analyses.**

905. Analyses may be made by the Engineer from finished material representing each melt. The chemical composition thus determined shall conform to the requirements specified in Article 903 for check analysis.

**Tension Tests.**

906. The material shall conform to the following requirements as to tensile properties:

| | |
|---|---|
| Tensile strength, in pounds per square inch | 80 000–95 000 |
| Yield point, minimum, in pounds per square inch | 45 000 |
| Elongation in 8 in., minimum, percentage | $\frac{1\,500\,000}{\text{Tensile strength}}$* |
| Reduction of area, minimum, percentage | 30† |

907. The yield point shall be determined by the drop of the beam of the testing machine when operated at a cross-head speed not greater than ½ in. per min. The testing machine shall not be stopped to obtain the drop of the beam.

**Modifications in Elongation.**

908. For material more than ¾ in. in thickness, a deduction of 0.25% from the percentage of elongation specified in Article 906, shall be made for each ⅛ in. of the specified thickness more than ¾ in.; but the reduced percentage shall not be less than 14 per cent.

**Modifications in Reduction of Area.**

909. For material more than ¾ in. in thickness, a deduction of 0.50% from the percentage of reduction of area specified in Article 906, shall be made for each ⅛ in. of the specified thickness more than ¾ in.; but the reduced percentage shall not be less than 24 per cent.

**Bend Tests.**

910. The test specimen shall stand being bent cold through 180° without cracking on the outside of the bent portion around a pin, the diameter of which shall have the following relation to the thickness of the specimen:

| Specimen Thickness. | Pin Diameter. |
|---|---|
| Up to ¾ in., inclusive | 1 times the thickness |
| More than ¾ in. to 1 in., inclusive | 1½ times the thickness |
| More than 1 in. to 1¼ in., inclusive | 2 times the thickness |
| More than 1¼ in | 2½ times the thickness |

**Test Specimens.**

911. Test specimens shall be prepared for testing from the material in its rolled or forged condition, except as specified in Article 912.

912. Test specimens for annealed material shall be prepared from the material as annealed for use, or from a short length of a full section similarly treated.

913. Test specimens shall be taken longitudinally, and, except as specified in Articles 915 and 916, shall be of the full thickness or section of material as rolled.

914. Test specimens for plates, shapes, and flats may be machined to the form and dimensions shown in Fig. 5, or with both edges parallel.

915. Tension test specimens for material more than 1 in. in thickness or diameter may be machined to a thickness or diameter of at least ¾ in., for a length of at least 9 in.

916. Bend test specimens for material more than 1¼ in. in thickness or diameter may be machined to a thickness or diameter of at least ¾ in., or to 1 in. by ½ in. in section.

917. The machined sides of rectangular bend test specimens may have the corners rounded to a radius not more than ⅛ in.

* See Article 908.

† See Article 909.

**Number of Tests.**

918. Two tension and one bend test shall be made from each melt; except that, if material from one melt differs $\frac{3}{4}$ in. or more in thickness, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

919. If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

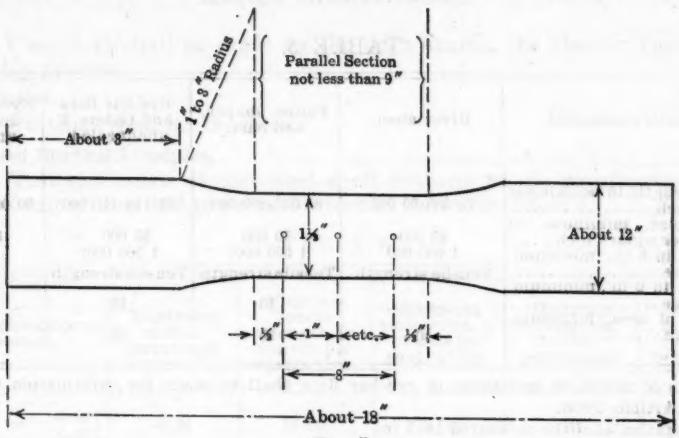



FIG. 5.

920. If the percentage of elongation of any tension test specimen is less than that specified in Article 906 and any part of the fracture is outside the middle third of the gauge length, as indicated by marks placed on the specimen before testing, a re-test shall be allowed.

**Additional Requirements.**

921. Articles 826 to 834, on Structural Steel, shall apply to Silicon Steel.

## SECTION 10.—NICKEL STEEL

**Process.**

1001. The steel shall be made by the open-hearth process.

**Discard.**

1002. A sufficient discard shall be made from each ingot intended for eye-bars to secure freedom from injurious piping and undue segregation.

**Chemical Composition.**

1003. The steel shall conform to the following requirements as to chemical composition:

| | Structural Steel. | Rivet Steel. |
|---|---|---|
| Carbon | not more than 0.45% | not more than 0.30% |
| Manganese | " " " 0.70% | " " " 0.60% |
| Phosphorus, Acid | " " " 0.05% | " " " 0.04% |
| Phosphorus, Basic | " " " 0.04% | " " " 0.03% |
| Sulfur | " " " 0.05% | " " " 0.045% |
| Nickel | not less than 3.25% | not less than 3.25% |

**Ladle Analyses.**

1004. An analysis of each melt of steel shall be made by the Manufacturer to determine the percentages of the elements specified in Article 1003. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus determined shall be reported to the Engineer or his representative, and shall conform to the requirements specified in Article 1003.

**Check Analyses.**

1005. Analyses may be made by the Engineer from finished material representing each melt. The chemical composition thus determined shall conform to the requirements specified in Article 1003.

**Tension Tests.**

1006. The material shall conform to the requirements as to tensile properties, given in Table 5.

TABLE 5.

| Properties considered. | Rivet steel. | Plates, shapes, and bars. | Eye-bar flats and rollers, ‡ unannealed. | Eye-bar flats* and pins,‡ annealed. |
|---|---|---|---|---|
| Tensile strength, in pounds per square inch | 70 000-80 000 | 85 000-100 000 | 95 110-110 000 | 90 000-105 000 |
| Yield point, minimum, in pounds per square inch | 45 000 | 50 000 | 55 000 | 52 000 |
| Elongation in 8 in., minimum percentage | 1 500 000 / Tensile strength | 1 500 000† / Tensile strength | 1 500 000† / Tensile strength | 20 |
| Elongation in 2 in., minimum, percentage | ..... | 18 | 16 | 20 |
| Reduction of area, minimum, percentage | 40 | 25 | 25 | 35 |

* Tests of annealed specimens of eye-bar flats shall be made for information only.

† See Article 1008.

‡ Elongation shall be measured in 2 in.

1007. The yield point shall be determined by the drop of the beam of the testing machine.

**Modification in Elongation.**

1008. For plates, shapes, and annealed bars more than 1 in. in thickness, a deduction of 0.25% from the percentage of elongation in 8 in., specified in Article 1006, shall be made for each $\frac{1}{32}$ in. of the specified thickness more than 1 in.; but the reduced percentage shall not be less than 14 per cent.

**Character of Fracture.**

1009. All broken tension test specimens shall show either a silky or a very fine granular fracture of uniform color and free from coarse crystals.

**Bend Tests.**

1010. Bend test specimens shall stand being bent cold through 180° without cracking on the outside of the bent portion, as follows: Material $\frac{3}{4}$ in. or less in thickness, around a pin the diameter of which is equal to the thickness of the specimen; and material more than $\frac{3}{4}$ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

1011. Bend test specimens for rivet steel shall stand being bent cold through 180° flat on themselves without cracking on the outside of the bent portion.

**Drift Tests.**

1012. Punched rivet holes pitched two diameters from a planed edge shall stand drifting until the diameter is enlarged 50% without cracking the metal.

**Test Specimens.**

1013. Test specimens shall be prepared for testing from the material in its rolled or forged condition, except as specified in Articles 1014 and 1015.

1014. Test specimens for annealed material shall be prepared from the material as annealed for use, or from a short length of a full section similarly treated.

1015. If the percentage of elongation of any tension test specimen is less than that specified in Article 1006 and any part of the fracture is more than

¼ in. from the center of the gauge length of a 2-in. specimen, or is outside the middle third of the gauge length of an 8-in. specimen, as indicated by scribe scratches marked on the specimen before testing, a re-test shall be allowed.

**Additional Requirements.**

1016. Articles 815 to 824 and 826 to 834, on Structural Steel, shall apply to Nickel Steel.

## SECTION 11.—CAST STEEL

**Process.**

1101. Cast steel shall be made by the open-hearth, the electric furnace, or the crucible process.

**Heat Treatment.**

1102. Castings shall be annealed.

**Chemical and Physical Properties.**

1103. Test specimens of cast steel shall conform to the requirements as to chemical composition and physical properties shown in Table 6.

TABLE 6.

| Maximum phosphorus, percentage. | Maximum sulfur, percentage. | Minimum tensile strength, in pounds per square inch. | Minimum yield point, in pounds per square inch. | Minimum elongation, in 2 in., percentage. | Minimum reduction of area, percentage. |
|---|---|---|---|---|---|
| Basic ..... 0.05<br>Acid ..... 0.06 | 0.06 | 60 000 | 30 000 | 24 | 35 |

**Ladle Analyses.**

1104. An analysis of each melt of steel shall be made by the Manufacturer to determine the percentages of carbon, manganese, phosporus, and sulfur. This analysis shall be made from drillings taken at least ¼ inch beneath the surface of a test ingot obtained during the pouring of the melt. The chemical composition thus determined shall be reported to the Engineer.

**Check Analyses.**

1105. Check analyses may be made by the Engineer from a broken tension or bend test specimen. The phosphorus and sulfur content thus determined shall not exceed that specified in Article 1103 by more than 20 per cent. Drillings for analysis shall be taken not less than ¼ in. beneath the surface.

**Yield Point.**

1106. The yield point shall be determined by the drop of the beam of the testing machine.

**Bend Tests.**

1107. The test specimen shall bend cold through 90° around a 1-in. pin without cracking on the outside of the bent portion.

**Test Specimens.**

1108. Tension and bend test specimens shall be taken from test bars cast attached to the castings where practicable. If the design is such that test bars should not be attached to the castings, the bars shall be cast attached to special blocks, of which a sufficient number shall be provided for each lot of castings. Test bars from which tension and bend test specimens are to be taken, shall remain attached to the castings or blocks they represent through annealing and until presented for inspection. Test bars shall be provided in sufficient numbers to furnish the tests required in Article 1111.

1109. Tension test specimens shall conform to the dimensions shown in Fig. 4. The ends shall be of a form to fit the holders of the testing machine in such a way that the load will be axial.

1110. Bend test specimens shall be machined to 1 in. by ½ in. in section, with corners rounded to a radius of not more than $\frac{1}{16}$ in.

**Number of Tests.**

1111. One tension test and, when specified, one bend test shall be made from each melt in each annealing charge and, when specified, from each casting weighing 500 lb., or more.

1112. If the percentage of elongation of any tension test specimen is less than that specified in Article 1103 and any part of the fracture is more than ¾ in. from the center of the gauge length, as indicated by scribe scratches marked on the specimen before testing, a re-test shall be allowed.

1113. If the results of the physical tests of any test lot do not conform to the requirements specified, the manufacturer may re-anneal such lot not more than twice and re-tests shall be made as specified in Article 1103.

**Workmanship and Finish at Foundry.**

1114. The castings shall conform substantially to the drawings. They shall be made in a workmanlike manner and shall be free from injurious defects.

**Inspection at Foundry.**

1115. Tests and inspection shall be made at the place of manufacture prior to shipment, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

**Rejection.**

1116. Castings which show injurious defects subsequent to their acceptance at the Manufacturer's works will be rejected and the Manufacturer shall be notified.

## SECTION 12.—CAST IRON

**Conformity to A. S. T. M. Specifications.**

1201. Cast iron, when required, shall conform to the A. S. T. M. Standard Specifications, A 48-18, for Gray-Iron Castings, and A 88-24, for High-Test Gray-Iron Castings.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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# PAPERS AND DISCUSSIONS

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## REPORT OF THE COMMITTEE OF THE IRRIGATION DIVISION ON "A NATIONAL RECLAMATION POLICY"

### Discussion*

BY MESSRS. JOHN A. WIDTSOE AND F. H. NEWELL.

JOHN A. WIDTSOE,† ESQ. (by letter).‡—The two supporting papers§ of this report contain views that will contribute much to the development of a successful reclamation policy. They do not mention, however, the most important fact connected with the Federal irrigation movement, namely, that land reclamation as a National activity, is rooted deeply in National policies, not politics. Federal reclamation is an expression of a twofold desire: To possess the entire country by full and profitable settlement and to establish more firmly the most important source of National stability—agriculture—by providing landless, land-loving men with farms. That is a National concern. National reclamation policies, therefore, must look to ultimate rather than to immediate results.

There is a veiled fear in the report of the Committee that Federal irrigation may interfere with private business, and that rigorous restrictions, therefore, must be set up. Such interference would be a misuse of the Reclamation Act. Federal reclamation activities should concern themselves primarily with projects that private enterprise cannot, or will not, undertake. In fact, this has been done. Few, if any, of the splendid major structures of the Bureau of Reclamation would have been built with private capital. The Government ventures have encouraged, not competed with, individual enterprises. The greatest recent private irrigation development in America has been inspired by, and coincident with, the growth of Federal irrigation. The wealth of experience gained on Federal projects, systematically gathered and published,

* Discussion of the report of the Committee of the Irrigation Division on A National Reclamation Policy, continued from November, 1929, *Proceedings*.

† Liverpool, England.

‡ Received by the Secretary, September 28, 1929.

§ *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, pp. 1193 and 1207.

is used profitably on private enterprises. Indeed, the Federal projects have not brought the Government into business, but rather have increased private business.

The present field for Federal irrigation is probably the completion and rounding out of existing projects, private and Governmental. Many private projects are unfinished and they represent large money losses. The reclamation fund could bring these into profitable operation, and salvage some of the losses. Other projects, established and profitable, face with dread the inevitable dry year. Insurance can be provided by a supplementary supply of water from high-priced, storage reservoirs, which can be built on the terms of the Reclamation Act, but are beyond the scope of ordinary banking practice. Then, also, small pioneering communities, "in the very raw," are scattered about in the desert, waiting for help—aside from their own toil—to construct the little reservoir and canal necessary to bring the meager reward of their efforts. Such enterprises are disdained by the banker; for such communities the Bureau of Reclamation can render the service clearly implied in the original Act.

Reclamation is not accomplished overnight; it requires periods of time beyond those recognized in the market place. Financial elasticity must be available for successful reclamation. Men cannot, under ordinary conditions, reclaim the desert in one year or in ten years. Men grow old and leave their sons after them to convert the dry desert into desirable homes. Governmental assistance must be given often at an immediate loss, but with ultimate gain. The form which this assistance took in the Reclamation Act was the use of interest money in the construction of the projects. The capital was to be repaid, but no more was expected of the water user. No better or fairer plan has yet been devised. The suggestion* of the Committee that the necessary assistance should appear in "the assumption of a part of the cost of project works" and that the water user should pay interest on the remainder, seems fraught with much danger and difficulty. Primarily, it invites loss. How much of the cost should be assumed? Should the first settlers, when the project is new and struggling, pay the same rate of interest as those who come on later, when much of the pioneer work has been done? It would be better to cling to the wise statesmanship of the original Act.

Lack of knowledge concerning the principles involved in reclamation by irrigation has led to considerable losses on Federal projects. Much has been learned since 1900. The errors of the past need not and will not be repeated. Looking forward in irrigation is the present problem. The past can be used to help in the solution of difficulties and not as a means to frighten later settlers.

Among the mistakes of the Federal irrigation venture, none is more glaring than the assumption that to provide water for land is the essence of a successful irrigation project. The human factor is ignored. The error, indeed, went further in the assumption that water was more important than land or man. Alkaline water on good land, or good water on land of low productivity, spells failure; the best of water on the best of land will yield no success unless the

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2098.

man, by inborn power and by necessary help, can use the combination of water and land profitably. The stern dictum of the future must be: (1) Do not impound or divert alkali water for irrigation purposes until Science can show how such water may be used safely; (2) do not build irrigation works for lands of low productivity, until Science has taught owners how to farm them profitably; (3) supply the water user (the man who is to combine water and land into a unit of civilization) with knowledge, tools, and money, if necessary; and (4) vary the annual repayment with the possible annual income. If the Reclamation Service is foolish enough to bring water to infertile lands, or to lands beyond favorable markets, then it must be content to let the water user have ample time in which to pay.

Mr. Mead's contentions are sound.* Successful irrigation cannot be expected in the future unless social and economic conditions are given first consideration. The best of engineering structures—ample water, fertile soil, and good markets—are demanded; but the man who uses all these factors, determines the success of the combination, and his needs, which are more difficult to meet than the stresses and strains of the dam, must be the concern of the new day in reclamation by irrigation.

One cannot have an intimate knowledge of conditions in Europe and take very seriously the cry of agricultural over-production through irrigation expansion. The relative area under the ditch is too small and the rate of increase in population is too great. With every coming decade men will look for more land, not for limitation of the agricultural area. Lands are reclaimed under the ditch, slowly, and the building of new irrigation works and the opening of new projects come with equal deliberation. It cannot be otherwise.

F. H. NEWELL,† M. AM. SOC. C. E. (by letter).‡—The value of a National Reclamation Policy must be judged by the practical, not theoretical, effect of such policy on the prosperity, the comfort, and the health of the Nation as a whole. It is to be assumed that the report of the Commitee of the Irrigation Division§ has this National, as contrasted with local, viewpoint in mind. If, for example, the drift of affairs should be such that the Federal policy, in promoting the reclamation of 1 000 000 acres, should result in the delay or should discourage the States, irrigation districts, or corporations, organized under State laws, from reclaiming 2 000 000 acres of better land, then there must be something wrong. Not that this is an established fact; the tendency is there.

The first point made in the report,‖ that of the unwisdom of waiving interest payments, emphasizes this tendency. Its effect is that of a subsidy, one by which less than 7% of the owners of irrigated lands, those on Federal projects, enjoy interest-free advances, while more than 93%, that is, land owners on State or corporate projects, must pay in full. To state it in another way; "It

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1207.

† Formerly Chf., U. S. Reclamation Service; now Pres., The Research Service, Inc., Washington, D. C.

‡ Received by the Secretary, October 26, 1929.

§ *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2097; also, May, 1929, Papers and Discussions, p. 1193.

‖ *Loc. cit.*, September, 1928, Papers and Discussions, p. 2098.

appears unfair that the farmers on 392 000 000 acres in the United States should be taxed for the purpose of supplying a large subsidy to those on only 1 803 000 acres, supplied with water by the United States." If this condition exists then the answer seems obvious as to why irrigation is not more advanced in many States and irrigation districts, and also why efforts are concentrated on getting more Federal appropriations. As long as kindly Uncle Sam, with his boundless wealth, will advance the money, it would be a waste of time to bother with other ways and means.

This easy credit, however, has not been an unmixed blessing. A reason assigned for the large percentage of failures of the farmers on Federal projects has been that these men, stimulated by the prevailing speculative spirit, have been less cautious; they have tried to handle 80 acres or 160 acres and, with lack of capital and experience, they have fallen deeply into debt. They might have fed their families, got less into debt, and acquired skill on smaller units.

The way in which this remarkable situation has come about, should be kept clearly in mind in evaluating the points of the Committee report. Most engineers have forgotten, if they ever knew, how these remarkable precedents of easy credits were established; how it happened that land owners on Federal irrigation projects are exempt from interest payments and form a class apart from other debtors of the Government. To get the full import of the Committee report, it is desirable to sketch briefly the points from which a start was made and the direction taken during the last quarter of a century in the development of the present policy.

When the Reclamation Act was signed by President Roosevelt on June 17, 1902, it had been preceded by nearly fourteen years of investigation initiated by Maj. John W. Powell under the authority of Congress (Acts of March 20 and October 2, 1888). Topographic and hydrographic surveys leading to the selection of various desirable projects, by 1901, were well under way. Thus, when Theodore Roosevelt became President it was possible to give him sufficient information to enable him, in his first message, to bring the needs of the country forcibly to the attention of Congress.

During preceding decades rapid progress had been made in irrigation; millions of acres of arid land had been irrigated by individual or co-operative effort; other millions of acres had been selected or purchased for reclamation. The rate of growth of irrigation at the beginning of the Twentieth Century, however, was slowing down. The economic reasons for this were not then fully understood. Attention was concentrated upon certain obstacles which might be removed by the Federal Government and which, if removed, it was believed, might stimulate further development by individuals and State aid.

The great waves of westward movement of people, which marked the Nineteenth Century, had begun to subside, but the impulse still remained; there was demand for more land to be opened. This was on the assumption that the pioneer spirit still continued. Here was a fallacy; the true spirit of pioneering had been overshawdowed by the spirit of speculation. What really dominated was the thirst for pioneer profits rather than for homes on the land. Appeals had been made to each of the Western States to recognize its

opportunities and responsibilities. As an entering wedge the Carey Act was passed in 1894 permitting the States to select public lands and initiate reclamation projects within their boundaries. Much enthusiasm was aroused, but the States did not rise to the opportunities; the conditions were such that under the Carey Act neither the real pioneer farmers nor the investors succeeded.

Without going deeply into the economic conditions, it was then assumed that a new impetus would be given to Western settlement if the Federal Government would take the lead in a series of experiments. Year after year unavailing efforts were made to obtain appropriations from Congress for building irrigation works. It was not accomplished until the rather ingenious scheme was devised to avoid making an appropriation. The result was achieved by diverting certain moneys before they became part of the general funds in the Treasury. This scheme circumvented the "watch-dogs of the Treasury" in the Committee on Appropriations.

It was urged that by the use of this special fund the Federal Government could build certain large works, the nature of which was such that they could not be undertaken by private capital, or even by the States, for the reasons either that the lands were owned by the Federal Government, or because there were certain interstate and international difficulties involved, or from the fact that the first costs were so large as to be practically prohibitive.

This condition should be kept in mind in order to understand the starting point of the National reclamation policy. In choosing among the large number of projects presented at that time, one of the first requirements was that any project to be considered should consist of public land; next that there should be some obstacle insurmountable by private agencies, such as State lines or of rights of a character such that they could be acquired only by the Federal Government. Moreover, it was a requisite that the enterprise should be of such nature that it was improbable that private capital could succeed.

At that time it was not a question of choosing the easier or more attractive projects; rather the reverse. Practically all the easy or cheap enterprises had already been undertaken; the various States had already selected, under the Carey Act, millions of acres including schemes which then seemed practicable. The requirement was that the Federal Government should undertake in each State one of the "rejects", or an enterprise so vast that private capital, even if aided by the State, could not hope for success; or it should confine its efforts to places where interstate and international complications existed. For example, one of the first determined upon was the Milk River Project, on the Canadian border, necessitating a treaty with Great Britain. Another was the Newlands Project, in Nevada, involving interstate relations in storing water and on which large areas of public land then remained.

It is possible to go down the list and indicate for each of the original enterprises the particular difficulty to be overcome and which formed the basis for selection. Some of these obstacles have been forgotten, but at the time they were the chief determining factors. In each case as careful a study as possible was made, not only of topography and hydrography, but

also of climate, soil, prospective crops, and markets, as well as of the economic features which might be expected. There was, of course, relatively little known at that time concerning the behavior of soils under irrigation, but the experts of the Department of Agriculture were consulted. They subsequently conducted experiment stations on the projects such as to make available a vast accumulation of facts for future guidance.

This National reclamation policy was gradually modified in detail as successive Secretaries of the Interior were called upon to act in specific cases. The principal changes in the original policy came when Secretary Lane, confessedly weary of endless importunities, took steps to put the responsibility upon Congress. Here was the breaking point from what might have been called the Rooseveltian policy of executive decisions. Up to that time the various Secretaries of the Interior, co-operating with the Secretaries of Agriculture, had selected the best available experts, appointed committees or commissions, and had acted on the advice of these engineering, agricultural, or economic boards. After Secretary Lane set aside this policy there resulted, in the opinion of many engineers, a sacrifice of scientific and technical expert opinion.

According to the ideals of the founders of the National reclamation policy, the results should be judged, not so much by the area of lands irrigated, nor by the total value of products, as by the influence for good upon individuals and associations of citizens. Federal reclamation has added 10%, or less, to the area of irrigated lands in the arid region. It has done this by overcoming what at the time seemed to be insurmountable obstacles and not by following the easy course. In studying the results of reclaiming this relatively small area of irrigated land, by the use of Federal funds, the question may well be asked whether the outcome has helped or hindered further developments. Has it aided in the prosperity of the 90% of land irrigated by other than Federal funds? Is the Federal Government carrying out its early promise of stimulating individuals, communities, and States to increased activities in building up the arid West, or the reverse?

The nation-wide publicity given the affairs of the beneficiaries of the National bounty has tended to obscure or minimize in the minds of the public the far greater activities of the ordinary agencies of reclamation. The situation, to-day, in this respect is illustrated by the condition in Palestine where the well being of the 10% of Jews impresses the world as being far more important than that of the 90% of Arabs. So in the mind of the public the prosperity of the West has appeared to be tied up with the 10% of land owners under Federal projects; the great growth in agriculture, industry, and transportation through the 90% of private and corporate effort appears to be forgotten. To this extent results are not good.

In another respect also those acts, sometimes regarded as Federal interference with States' duties and opportunities, have the result of delaying various enterprises. It has been argued that the defects of the Carey Act would have been quickly remedied if hopes had not been pinned to the Federal Reclamation Act. This was passed at about the time that financiers and statesmen were discovering remedies for the difficulties encountered by

State administration of irrigation enterprises. It is argued that instead of stimulating the States—as had been hoped when the Federal Government began to overcome physical obstacles, as well as interstate and international difficulties—the reverse took place; the State officials, instead of emulating these efforts, sat back to see the outcome.

Criticism has been made of the waiving of interest payments to be made by the owners of land which have been reclaimed and made valuable by the use of Federal funds. At the time that these payments began to fall due, the idea was disseminated (and not discouraged) by Secretary Lane, that the rich and kindly Uncle Sam would not oppress any land owners; on the contrary he would grant them every possible favor. This attitude was in effect an invitation, especially when coupled with the statement, made on the floor of Congress at the time of the passage of the Reclamation Act, to the effect that the land owners would repudiate these debts. Thus, there came about an active campaign, not of outright repudiation, but for an indefinite extension of time. The effect of this was not confined solely to debts owed by land owners to the Government; it was seriously argued that other wealthy institutions, banks and loan companies, should also be generous. On certain Government projects the spirit of repudiation was so strong that financial houses hesitated or declined to do business in the localities, because of a general lowering of moral or financial standards due to the encouragement laid at the doors of Federal officials. It is not to be inferred that there was not real hardship in many cases. Under the present system, however, full discretion cannot be used by Bureau officers. All debtor land owners must be treated alike; hence, the breakdown.

Remember in this connection that the Secretary of the Interior deals with the people on the project, not as farmers, but as land owners; and also that a large proportion of these land owners (debtors of the Government) do not cultivate the land themselves; they are non-resident landlords. Some are bankers, business and professional men, clerks, and laborers trying to earn a living in town and renting the land to tenants. Due to these conditions the burden laid on the back of the actual cultivator of the soil often has not been diminished by executive leniency. Because of these easy terms given to the land owners and resulting speculation, the land prices have sharply advanced. The man who actually works in the fields has not been benefited.

The point which immediately concerns engineers, economists, and other citizens is as to whether the States, the irrigation districts, and corporations have been stimulated by the Federal activities. It may be asked whether these agencies might not have been more effective if the Federal Government had kept out of the field; the course of events might have forced the States to attack the reclamation problem through the terms of the Carey Act or some needed modification. All this is hypothetical, but leads to the conclusion that, with the experience of a quarter of a century, there should be a modification of Federal reclamation policy along the lines proposed by the Committee and one that will bring about a larger and better prosperity through encouraging these State agencies.

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PAPERS AND DISCUSSIONS

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# RELATION OF THE LANDSCAPE ARCHITECT TO THE ALLIED PROFESSIONS ENGAGED IN CITY PLANNING

## Discussion *

By Arthur A. Shurtleff, Esq.†

Arthur A. Shurtleff,‡ Esq. (by letter).§—In the planning of cities and towns the part played by engineers will always be one of the most important. The landscape architect, the architect, the sculptor, the financier, and the men who know the law cannot be left out, however, without injury to the city plan both at the beginning and during the progress of the development. If any one of these men plays too large a part at the expense of the others, the design of the city will suffer either in good appearance, convenience of use, financially, or in the mechanical operation. Evidently, the wit, devotion, or inspiration of one man may help a city vastly by starting an excellent plan in the right direction, but the value of such a man to a community increases as he gathers about him men representing all the professions and the arts.

The day has passed when any man of good judgment would attempt to plan an entire city single-handed. To-day, most communities are not inclined to trust a single individual, unless he has the genius to bring all the other needed minds into the work and to give these minds full scope for their abilities and talents. A man who works alone handicaps himself and the community. The ideal man is one possessing the best training, large experience, generous co-operative instincts, and ability to work constructively with all the many men who must participate in the designing of modern communities. A man like this may be an engineer, an architect, a statesman, or a landscape architect; the important element is co-operation.

* Discussion of the paper by Arthur A. Shurtleff, Esq., continued from May, 1929, *Proceedings*.

† Author's closure.

‡ Landscape Archt., Boston, Mass.

§ Received by the Secretary, October 16, 1929.

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## ELASTIC EQUILIBRIUM IN THE THEORY OF STRUCTURES

### Discussion *

By H. S. Richmond, Esq.†

H. S. Richmond,‡ Esq. (by letter).§—One of the discussers, Mr. Feld, brings forward three questions having to do with stress distribution which he states‖ the writer makes no attempt to discuss. The treatment of this subject in detail lies outside the scope of the paper. As indicated by Professor Timoshenko,¶ it is assumed in treating problems in elastic stability that failure is to occur, not through over-stress, but by excessive distortion below the yield point. The analyses are based upon relations prevailing below the elastic limit.

Recognition, however, is given in the paper to the weakening effect of a load similar to the crippling load, but less in value; but the designer must work out the stress relations in the particular design, making proper application of the theorems.

The writer's value for the torsional resistance functions of I and H-sections are only tentative. In theoretical studies, it should be recognized that this function not only varies from the polar moment of inertia, but that for these types of section, it actually has no relation thereto.

The discussion by Messrs. Seely, Putnam, and Schwalbe** is exceedingly interesting in illustration of the experimental determination of the "shear-center" in channels. The writer has not seen any discussion of this principle in the standard works on structural mechanics. Students have been free to arrive at all sorts of erroneous conclusions as to the action of channels as flexural members.

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* Discussion of the paper by H. S. Richmond, Esq., continued from August, 1929, *Proceedings*.

† Author's closure.

‡ With Gibbs & Hill, Cons. Engr., New York, N. Y.

§ Received by the Secretary, October 8, 1929.

‖ *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 1079.

¶ *Loc. cit.*, August, 1929, Papers and Discussions, p. 1647.

** *Loc. cit.*, April, 1929, Papers and Discussions, p. 1081.

Messrs. Julian and Rich* have made an interesting analysis and have prepared a working formula which, for the practical range which lies safely within the limit of the curve for the crippling load and for low slenderness ratios, meets an arbitrarily assigned working stress. It is to be noted that their formula, Equation (53),† is based upon the condition, $a = 0$, and that their working formula assumes this; therefore, care should be taken to see whether that condition prevails. In many problems, this is an important consideration.

It would be possible to derive a working formula for any given class and use of beam; the writer, however, would prefer not to crystallize the theorems into working formulas at this time, but would express the hope that the principles brought into consideration should receive further study and in due course be given recognition in the drafting of codes of design, especially where light and open-work types of construction are concerned.

The writer is obligated to Mr. Parcel‡ and other commentators§ for citations of analyses by previous investigators, showing points of correspondence with his own work.

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1285.
† *Loc. cit.*, p. 1288.
‡ *Loc. cit.*, August, 1929, Papers and Discussions, p. 1643.
§ *Loc. cit.*, April, 1929, Papers and Discussions, p. 1087.

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## THE GRAPHICAL SOLUTION OF A CORRELATION TABLE

### Discussion *

By Weston S. Evans, Esq.†

Weston S. Evans,‡ Esq. (by letter).§—The trend of the discussion seems to indicate that the writer considered the graphical solution to be superior to the arithmetical solution for solving correlation tables. This is not the case. Mr. Dunlap‖ was correct in stating that, "the advantage of the graphical solution is in the understanding and grasp derived of the problem of correlation while learning about this method of analysis."

If a numerical answer only is sought, plain arithmetic is the best method by which to obtain it. Means and standard deviations are very easily obtained graphically and if it becomes necessary or desirable to produce a finished drawing to scale, the extra time required to complete the solution is not great. The four hours required to solve the table in Fig. 2¶ was the time necessary to draw Fig. 12.** If the finished drawing is not required, the graphical solution is entirely out of place and 20 min. is sufficient. The writer has solved many similar tables in that length of time, after making the table which is a part of the graphical solution. If speed is the controlling factor, then punching, sorting, tabulating, and calculating machines are the implements which should be used. The average engineer, however, has many lessons to learn before he is able to use such devices in solving his statistical problems.

Mr. Dunlap states that it is better to assume the origin as near as possible to the center of gravity of the table than near one corner to avoid negative

* Discussion of the paper by Weston S. Evans, Esq., continued from August, 1929, *Proceedings*.

† Author's closure.

‡ Asst. Prof., Civ. Eng., Univ. of Maine, Orono, Me.

§ Received by the Secretary, October 9, 1929.

‖ *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1293.

¶ *Loc. cit.*, January, 1929, Papers and Discussions, p. 50.

** *Loc. cit.*, p. 55.

numbers.* For pencil and paper solution this is probably correct. The writer has been accustomed to adding machines on which the negative sign becomes a source of trouble and the magnitude of the numbers is of little consequence. Furthermore, when the data are tabulated on cards for use in sorting and tabulating machines, the code for any variable must start with the lowest values, and hence the origin will be in the lower left-hand corner. Although the choice of origin is optional, the writer believes that the best practice is to avoid negative signs.

The explanation of the regression equations as given by Mr. Strachan† is worthy of some elaboration. In the two regression formulas, $x = 0.90\,y + 11.59$, and $y = 0.90\,x + 9.74$ (Equations ($5a$) and ($7a$)),‡ it becomes evident that although both apply to the same data, a given value of $y$ gives different values of $x$. For example, if $y = 100$ in Equation ($5a$), $x$ will equal 102.22; but it cannot be said that if $x = 102.22$, $y$ will equal 100, since, from Equation ($7a$), $y$ would equal 101.00. As $r$ decreases, the difference between the two equations will increase. In Equation ($5a$), $x$ is the mean of all 7-day ratios that will occur for any given 28-day ratio; while in Equation ($7a$) $y$ is the mean of all 28-day ratios that will occur for a given 7-day ratio. This principle, so seldom applied to engineering data, is one of the best answers to the question, "Why a correlation table?"

The discussions by Messrs. Moyer† and Hatch§ are somewhat aside from the problem of solving a correlation table, but are nevertheless very interesting. Mr. Hatch has presented very well the possibilities of partial and multiple correlation.

Mr. Jakobsen‖ has well expressed the value of determining the probable error of observations, estimates, and mathematical conclusions. The writer fully agrees that such statistical methods as have been discussed, if applied to many problems relative to engineering work, would produce definite conclusions rather than long drawn-out discussions which frequently lead to nothing.

In closing, the writer wishes to emphasize that means, standard deviations, and correlation coefficients are constants which are necessary to establish facts in regard to the relationship existing between pairs of variables. For simple relationship between two variables the simple correlation coefficient is satisfactory; but as relationships become intricate and many, the partial and multiple coefficients, as outlined by Mr. Hatch, will relate facts that can be obtained in no other way. Whether the correlation table is solved graphically or arithmetically, its use in the engineering field should become more common and thereby give a better understanding of the relationship between cause and effect.

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* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1294.
† *Loc. cit.*, p. 1295.
‡ *Loc. cit.*, January, 1929, Papers and Discussions, p. 56.
§ *Loc. cit.*, May, 1929, Papers and Discussions, p. 1299.
‖ *Loc. cit.*, August, 1929, Papers and Discussions, p. 1653.

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# USE OF WATER ON FEDERAL IRRIGATION PROJECTS

## Discussion *

By George N. Carter, M. Am. Soc. C. E.

George N. Carter,† M. Am. Soc. C. E. (by letter).‡—The author has assembled and presented a vast amount of data gleaned from actual irrigation practice rather than from theory.

To one well acquainted with the project to which any one of the detail twenty-four tabulations§ may relate, the information is especially valuable. However, to apply these figures to the design of a new project, without intimate knowledge of all the details which affect the demand and use of irrigation water, quite likely might result disastrously. It is just as dangerous to project from such data into the development field as to apply wrongly any highly involved engineering formula in design.

It is to be noted that the use of water varies widely, between the limits of 0.65 acre-ft. per year (the minimum on the Milk River Project, in Montana) and 7.01 acre-ft. per year (the maximum on the King Hill Project, in Idaho). Between these limits are found two projects that are successful and well-balanced in every respect, the Yakima (Sunnyside Division), in Washington, and the Boise, in Idaho, with an average annual use of 3.29 and 3.60 acre-ft., respectively.

By the expression, "well-balanced," is meant that soil conditions are normal, if there is such a thing as normal soil; that temperature and growing season are conducive to maturing numerous and diversified crops, indigenous to the temperate zones; that no reliance may be placed on rainfall and plant growth is entirely dependent on irrigation; and that the physical irrigation system is well designed and permits of good distribution and control of the water available.

* Discussion of the paper by E. B. Debler, M. Am. Soc. C. E., continued from August, 1929, *Proceedings*.

† Commr. of Reclamation, Boise, Idaho.

‡ Received by the Secretary, October 7, 1929.

§ *Proceedings*, Am. Soc. C. E., March, 1929, Papers and Discussions, pp. 753–776, inclusive.

Factors which contribute to the wide variations are explained, but it would have been quite helpful if the author could have enlarged upon his description and classification of soils of the various projects. For example, on the King Hill Project, the use of water (see Table 6)* greatly exceeds that of other projects and may appear excessive and wasteful to one unacquainted with conditions; but a brief description of the predominating soil characteristic is quite illuminating. The top-soil, averaging from 2 to 3 ft. in depth, is light and sandy, and is underlaid by Snake River black lava sand to great depths. The water-consuming possibilities of this soil and subsoil are obvious; likewise is the problem that such a condition imposes upon the farmer. Irrigations must be very frequent. The writer examined a field of alfalfa seeding on the King Hill Project, planted with a nurse crop of wheat, and found the young alfalfa plants wilting six days after thorough irrigation. Furthermore, the application of such large quantities of water is rapidly leaching all plant food elements out of the soil, as demonstrated by the fact that each year the use of water is increasing, while crop yields are gradually decreasing.

The methods of securing field data and reducing them to as accurate forms as possible, are in keeping with good practice. The Twin Falls North Side Land and Water Company while constructing a 185 000-acre Carey Act Irrigation project in Idaho, carried on for five years what was perhaps as comprehensive and thorough a study of the use of water for irrigation as has ever been undertaken in the United States. The under-measurement of water delivered to the farmer was found to be 5%, occasioned in large part by the tendency of the Cipolletti weir to under-read due to velocity of approach, caused by silting of weir pools. It was found more satisfactory to calibrate the weirs in their actual operating condition, employing expert hydrographers for the purpose, than to attempt to maintain them in ideal physical condition.

The author finds† the expression "loss in feet of depth per day" over exposed water surface or wetted perimeter "impractical." It occurs to the writer that this is a most rational measure of canal loss. Expressed on this basis, a unit is afforded which may be applied to any operating condition between peak load and no load. A canal system may carry full load with a net delivery of 70% and half load with net delivery of only 40 or 50 per cent. Under these two conditions, the loss in feet of depth per day will be practically the same, while the percentage of loss varies greatly. Wide, shallow, low-velocity canals will have a much higher percentage loss than when the same quantity of water is conveyed in a deeper and higher-velocity channel, while the loss in depth of feet per day over wetted perimeter of exposed water surface is the same for either type of canal, all other conditions being equal.

Some elaboration might well be made as to the quantities of waste expressed as "Percentages of Total Diversions for Seasons", in Tables 1 to 24. An efficiently operated canal system should not waste out of the lower end much more than 4 or 5% of the total supply. Through careful operation, this figure is often reduced to 2 or 3 per cent. Any waste of more than 4% is ordinarily avoidable. Therefore, in using the author's data on this item, it may be under-

* *Proceedings*, Am. Soc. C. E., March, 1929, Papers and Discussions, p. 758.
† *Loc. cit.*, p. 779.

stood that the large percentages are voluntary and need not be admitted as a necessary part of the available supply.

The collection and intelligent application of data such as the author has presented, are invaluable in a consideration of any irrigation project, especially a large one. Owners and operators quite frequently under-estimate the value of such a survey and hesitate to appropriate funds for the purpose. Nevertheless, the characteristics of the various parts of the water-distribution system, with respect to the use of water, as revealed by detail hydrographic records, will locate places of excessive loss and point the way to a remedy, thus saving many times the cost of such work.

Given favorable soil and climatic conditions, the prosperity of an irrigation project is reflected almost entirely by the quantity of water it has available. Despite extravagant and wasteful use, water-logging of land, and other difficulties ordinarily credited to over-use of water, the project's well-being and financial success will be in proportion to the quantity of water used. Land values, diversity and intensity of farming, and crop yields, all go down in proportion as the available water supply diminishes, and almost without exception the greatest use of water is found to be coupled with the most prosperous project.

The popular and numerous investigations undertaken throughout the irrigated West about twenty years ago to determine the duty of water have been harmful. They were predicated on laboratory methods rather than actual field conditions, and they aimed at a result which would be the minimum, in order that the irrigated acreage of the Western States might be extended to a maximum. Knowledge of the conditions under which the farmer would perpetually be compelled to till these extended acres was lacking. The experience of the last twenty years of irrigation activities has well demonstrated the fallacy of the original object.

In the beginning of irrigation on a large scale throughout the Western States it seemed desirable to produce a maximum of crop per unit of water, but now it is evident that the maximum per unit of land is the only basis upon which the irrigation farmer can remain. Otherwise, taxes, overhead, labor, and equipment costs, carry the farmer to the point of "Diminishing return" and frequently beyond it.

Witness the voluntary efforts and expenditures of many co-operative irrigation companies to enlarge their water supplies after they have received control of their projects from the builders. They have learned, during the years of pioneering, that only with a generous and dependable water supply can they expect to prosper.

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PAPERS AND DISCUSSIONS

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# WATER SUPPLY FROM RAINFALL ON VALLEY FLOORS

## Discussion *

By W. P. Rowe, Assoc. M. Am. Soc. C. E.

W. P. Rowe,† Assoc. M. Am. Soc. C. E. (by letter).‡—The question of consumptive uses of various types of vegetation is one that is being studied by a great many investigators. These investigations, as a rule, are being made in connection with studies not directly allied with engineering but which, when finally assembled and correlated, will prove of great value to the engineer dealing with water supply.

The author has spent considerable effort in determining the consumptive use of the vegetation on various water-sheds in Southern California. Of all the water-sheds given in Fig. 6§ and Table 11,‖ Devil and Waterman Canyons are the only ones which have a brush cover similar to the valley floor area, the others, reaching to higher elevations, have considerable areas of pine and oak timber. The author states¶ that "the Valley of San Bernardino is a sloping plain and for the purpose of this illustration is assumed to have a natural brush cover." This classification eliminates from the discussion all the water-sheds except Devil Canyon and Waterman Canyon.

Transpiration losses from these two water-sheds occur from two distinct types of vegetation. The canyon bottom with its concentration of escaping seepage water in the form of underflow and surface streams supports a water-loving growth in which alders predominate. The area above the canyon bottom supports a vegetation that is Nature's solution to the problem of a water-shed cover that will take water when it can get it and live through severe droughts. This is the true chaparral of dwarf forest. It will coppice readily after being burnt over and in a few years the cover is as thick as it was before the fire. Without

* Discussion of the paper by A. L. Sonderegger, M. Am. Soc. C. E., continued from October, 1929, *Proceedings*.

† Cons. Engr., San Bernardino, Calif.

‡ Received by the Secretary, August 26, 1929.

§ *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1153.

‖ *Loc. cit.*, p. 1161.

¶ *Loc. cit.*, p. 1158.

this faculty of restoration, it would have been discarded by Nature thousands of years ago.

Any one familiar with the summer streams in the low altitude of Southern California, in which class Devil Canyon and Waterman Canyon belong, knows that there is a great variation between the quantity flowing in the morning and in the evening. This variation is caused almost entirely by the fluctuation in transpiration of the canyon-bottom growths, the mountain-side transpiration fluctuations being "ironed out" by the time and distance elements of percolation. In Devil Canyon the floor will fluctuate as much as 1.0 sec-ft., the morning flow being 1.5 sec-ft. and the evening flow, 0.5 sec-ft. This is not a measure of the total transpiration losses from the entire stream-bed growth because the time for the surface water in the stream to pass through growth has a tendency to overlap the fluctuations. The writer has found that alders on the desert side of the San Bernardino Mountain have a consumptive use of 8 acre-ft. per acre per annum, which is probably greater than the losses from alders under the cooler climate as given by the author* for the Arrowhead water-shed.

The writer fails to check the author's rainfall data, as shown on Fig. 8.* The elevation of the gauging station on Devil Canyon is about 1 800 ft. above sea level. The crest of the Devil Canyon water-shed meanders for a distance of about 4½ miles at an average elevation of 5 000 ft. The air-line distance across the canyon between the 5 000-ft. contours on either side is about 2½ miles. The water-shed is a typical fan-shaped one with its narrowest part at the gauging station. In Table 8,† the rainfall for Squirrel Inn, at the summit of the mountain, but easterly of Devil Canyon, is given as 73.58 in. for 1921-22 and 44.27 in. for 1922-23. The rainfall for Devore at about the same elevation as the gauging station, but westerly thereof, is given as 59.99 in. for 1921-22 and 30.76 in. for 1922-23. The author omits the Arrowhead Springs rainfall record. This station is east of the Devil Canyon gauging station and had a rainfall of 39.49 in. in 1921-22 and 19.58 in. in 1922-23. It is about the same elevation as the Devore rainfall station and the Devil Canyon gauging station. The average of the Arrowhead Springs and Devore records would give the approximate rainfall at the lowest and narrowest end of the Devil Canyon water-shed. This would make the 1921-22 and the 1922-23 rainfall at the Devil Canyon gauging station 61.66 in., and 34.72 in., respectively. If these are averaged with the Squirrel Inn record to obtain the average rainfall for the Devil Canyon water-shed, the result is 61.66 in. for 1921-22 and 34.72 in. for 1922-23. This is probably conservative as the width of the water-shed increases as it approaches the crest where the rainfall is greatest.

When these figures of average rainfall for the Devil Canyon water-shed are compared with the total run-off, the total consumptive use of the water-shed vegetation is seen to be 40 in. for the wet season of 1921-22 and 27 in. for the dry season following. There was probably some hold-over seepage water from 1921-22 which was available for plant life in 1922-23, so that the average consumptive use of 33 in. for the two seasons would be representative

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1156.
† *Loc. cit.*, p. 1159.

of what the vegetation could consume. The brush near the tops of the ridges will transpire less water than the brush farther down the slopes because the seepage water has a tendency to escape down hill where it becomes available for growth on the lower areas after the upper area is deficient in moisture. The brush on the northern exposures, being in the shade a longer period of time, will require less moisture than that on the hot southern exposures. It will be seen from this that the average consumptive use of the cover for the entire water-shed above the canyon bottoms is made up of brush having sub-normal and abnormal transpiration losses. The maximum possible consumptive use of the brush will be more than 3.0 acre-ft.

The outwash slopes of Devil and Waterman Canyons are composed of sands, gravels, and boulders with a top-soil supporting a heavy growth of chaparral, chamise predominating. This growth is neither as large nor as luxuriant as that on the northern exposures within the two water-sheds, as the southerly exposures of the outwash slopes offer maximum conditions for transpiration losses. The writer was instrumental in having installed a set of overhead sprinklers on a test area at the mouth of Devil Canyon by means of which the moisture available for plant growth could be augmented by artificial rains. Experiments are now being conducted by the Department of Agriculture, under the direction of Henry F. Blaney, Assoc. M. Am. Soc. C. E., to determine the consumptive use of this brush.

The roots of the native chaparral extend to a depth of 18 ft. Water was applied to the test plot through the sprinklers during the night hours in the spring when the air was cool and there was no wind. The quantity falling on the area was determined by means of rain gauges scattered over the area. An application of water at the rate of 1 in. per hour for 4 hours produced no run-off. The author has pointed out that, when comparing mountain growth chamisal with outwash slope chamisal, it must be remembered that in the mountain growths the root zone is limited by the depth to bed-rock, whereas in the outwash slopes the root zone is limited by the depth of rainfall penetration over an average of several years. The deep penetration of rainfall during a wet year will, no doubt, stimulate the root growth and encourage it to extend deeper, but the dry years with a lack of deep penetration will cause the deeper roots to become dormant. Any curtailment of chaparral growth on the porous outwash slopes will result in an additional quantity available for deep penetration to the underground water plane. After the deep-rooted native brush is removed and the area cultivated, but later abandoned, the native annuals soon overrun the area. These annuals have a depth of root zone of about 8 ft. which, when compared with the deep-rooted chaparral, should result in a lessened consumptive use by the weeds and a consequent greater supply to the underground water. In a region where at least 95% of the water supply is obtained from ground-water sources, this item becomes of great importance.

Fig. 14 shows the effect of fire on the chaparral cover in the mountain areas. Devil Canyon, with an area of 6.16 sq. miles above the gauging station, adjoins Waterman Canyon with an area of 4.55 sq. miles. These two canyons have identical exposures, and altitudes, are typically fan-shaped and

probably have equal rainfall. One of the main traveled roads to the mountain resort area in the San Bernardino Mountains traverses the length of Waterman Canyon. There have been frequent fires in the Waterman Canyon area, while Devil Canyon is more isolated and, prior to 1924, had had no serious fires for at least thirty years. In 1911, the Waterman Canyon area was almost entirely burned over. The stream-bed growth, however, generally escapes these fires, either because of its position in the bottom of the canyon, or, because of a higher moisture content in the vegetation, or perhaps a combination of both. The cover in the Devil Canyon water-shed escaped burning by the fire of 1911.

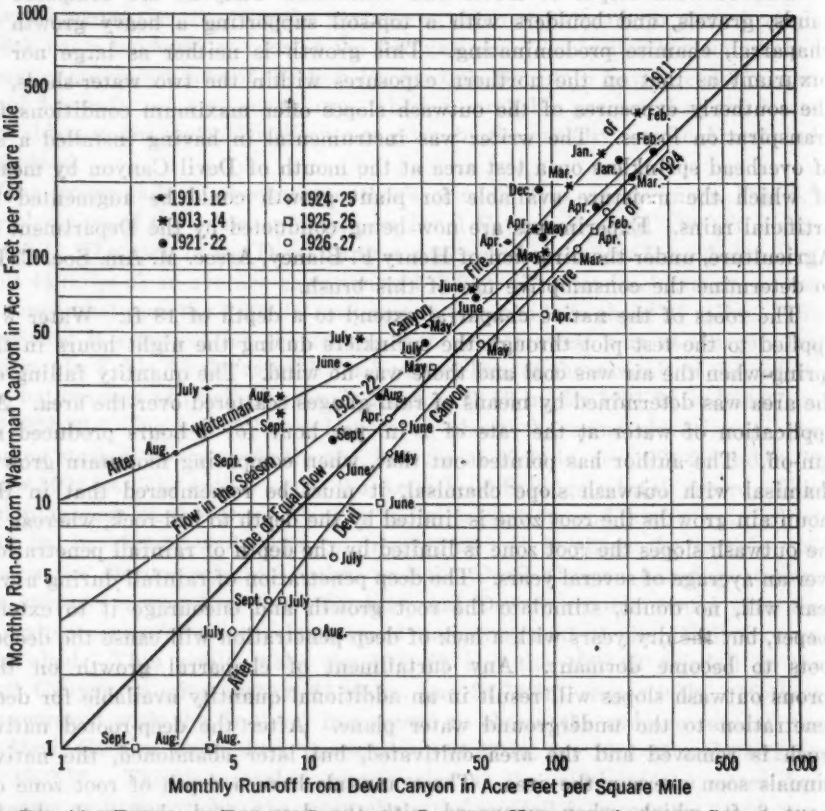


FIG. 14.—EFFECT OF FIRE ON CHAPARRAL COVER IN MOUNTAIN AREAS IN CALIFORNIA.

Stream-flow observations were made on both canyons for a few years following the Waterman Canyon fire. A comparison between the stream flow of Devil Canyon and Waterman Canyon is shown in Fig. 14. The low flows in winter are omitted from the diagram to avoid confusion with summer flows which are important. It will be noticed that the flow from Waterman Canyon exceeded that from Devil Canyon throughout the year. The wet season of 1913-14, three years after the fire, produced 515 acre-ft. per sq. mile more

run-off than Devil Canyon, the total for Waterman Canyon being 1 322 acre-ft. per sq. mile and for Devil Canyon 807 acre-ft. per sq. mile.

Inasmuch as the summer flow of Waterman Canyon was sufficient to supply the needs of canyon-bottom vegetation and yielded a surplus over the Devil Canyon flow, this excess of 515 acre-ft. per sq. mile, or 0.8 acre-ft. per acre, must be due to the decreased evapo-transpiration losses caused by the burning of the brush cover. The next season of heavy stream-flow record is 1921-22, which is also shown in the diagram. This curve approaches the line of equal run-off of both water-sheds, but is still on the Waterman Canyon side probably because of several small fires on that water-shed, which in the aggregate accounted for a considerable area of burned brush cover between 1911 and 1921.

In 1924 a fire burned over about one-half the Devil Canyon water-shed. A curve has been drawn from the stream-flow data since 1924. The Devil Canyon discharge has been corrected for diversions to the City of San Bernardino. This last curve falls on the Devil Canyon side of the line of equal flow. The differences between the curves for Waterman Canyon and Devil Canyon in Figs. 6 and 10* are due to the burning of the brush on the Waterman Canyon area. In this region where water is so valuable it appears to the writer that brush fires, which are the greatest menace to the timberlands in the higher area, and which excite the professional conservationist to hysteria and furnish the local newspaper with scare-heads rivaling those of some major action in the World War, are far from being an economic waste. All that remains is to put the excess run-off into the ground. Nature, by experimenting with this problem for thousands of years, has provided the débris cones or outwash slopes below the mouths of the canyons for this purpose.

The consumptive use of the brush cover on the outwash slopes is the determining factor in deep penetration of rainfall on the valley floor. Given the rainfall on the valley floor, and the consumptive use of the cover, the difference will be deep penetration. The run-off from the brush-covered valley floor is negligible. The author, after determining the requirements of the brush cover for the Devil Canyon water-shed on Fig. 8, ignores this factor when considering the valley floor area and attempts to solve the problem by utilizing his knowledge of seepage and storm water escape from the mountain area and by applying it to the valley floor area. Table 10† is a set-up of this comparison.

The author uses his Waterman Canyon curve to obtain rainfall-run-off relations. His Devil Canyon curve would have been better suited to conditions, as it retained its natural brush cover until the fire of 1924. Column (3) of Table 10 shows the seasonal rainfall to be expected from storms, as given in Column (2). If the author had applied his consumptive use curve as shown in Fig. 8 to this seasonal rainfall, he would have found that there would have been left for deep penetration and run-off on the valley floor an

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1157.
† *Loc. cit.*, p. 1160.

average of 3 in., instead of 16 in., for the three storms given for the San Bernardino Valley in Column (2) of Table (10).

The author does not distinguish between the effect of storms occurring in the winter months of December, January, and February, when native chaparral growth is practically dormant, and the effect of storms during the months of March, April, and May, when the chaparral is making its most active growth due to the ideal conditions of long warm days and high soil moisture content. If he had determined the relation between the consumptive use of the chaparral on the mountain areas and that of the valley chaparral this question would have been taken care of in the comparison. The greatest difficulty in the way of such a comparison would be the effect of hold-over seepage water in the mountain areas after a wet season.

When the Department of Agriculture has completed its study of the consumptive use of native plants on the San Bernardino Valley floor, it may be found that this is not such an important item as the writer believes. The author has offered the basis for a comparison between consumptive uses of mountain and valley-floor brush, which can be used in the future without resorting to the elaborate continuous field investigations required by the Department of Agriculture. The writer is convinced that there is a very important contribution to the underground water supply in Southern California from deep penetration of rainfall on the valley floor, but he can not conceive of such a figure as 250 acre-ft. per sq. mile per year as an average for the brush-covered lands of the San Bernardino Valley. The writer believes that the greatest value of such a study was utilized when areas that were formerly brush-covered became cultivated with crops and orchards. As a result of this improvement the transpiration losses from the native growths that would draw on the rainfall were conserved and utilized for the growth of valuable crops and the creation of taxable wealth. Studies made by J. B. Lippincott, M. Am. Soc. C. E., in the Santa Ana Valley, and by Harold Conkling, M. Am. Soc. C. E., in the San Gabriel Valley, show that the substitution of cultivated crops for brush-covered areas has made very little inroad on the total water supply as it was conceived to be at the beginning of the Twentieth Century. The future benefits of any knowledge of rainfall penetration on the valley floor will be derived by increasing this quantity through removing the brush-cover, or, substituting a shallower rooted growth. The brush on the mountain sides is absolutely essential and should not be destroyed, but there is the possibility of some systematic burning to remove the fire hazard and conserve excessive transpiration losses.

*Determination of Penetration from Soil Moisture Tests on North Mesa, Murietta-Temecula Area.*—Table 4* shows the specific gravity, porosity, and probable yield of two soil samples at Test Plot No. 1, one from a depth of 2 ft., and the other from a depth of 4 ft. The increase in specific gravity, decrease in porosity, and decrease in probable yield for the sample from a depth of 4 ft. would indicate a marked change in soil structure. The author states that† "the region is comparatively arid and the soil is an ancient, rather compact

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1146.

† *Loc. cit.*, p. 1145.

alluvium, a combination which would lead the casual observer to the conclusion that no deep penetration could occur." The writer made a casual observation of the region while conducting an unsuccessful search for Test Plot No. 1 as shown on Fig. 4,* and noticed a heavy hardpan underlying the top-soil wherever erosions had revealed the soil to a depth of 4 ft. The existence of hardpan is the best indication of the depth of rainfall penetration of which the writer has knowledge. The hog wallows and puddles mentioned by the author† indicate that there is no free penetration of water, at least in the upper soil zone.

An examination of the data presented in Table 5‡ shows that the soil under Test Plot No. 1 is either very heterogeneous or that some artificial factors were introduced. Column (6) of Table 5 shows that the soil for all depths below the fifth foot was below field capacity on March 7, 1927. This fact would indicate that either the root zone of vegetation extends to a greater depth than 15 ft., or that the data given in Column (2) are not typical for the entire test plot. Any one comparing Columns (7) and (2) of Table 5, is led to believe that there was no deep penetration whatsoever, because the soil moisture at all depths to 15 ft. was less than the field capacity. The building of dikes or levees around the test plot would create a condition that would prevent the escape of moisture which otherwise might disappear as run-off.

The data presented by the fluctuations of wells given in Table 7 are far from conclusive evidence that the fluctuations were due to direct penetration of rainfall. The data for Well No. 129 shows that, despite the heavy rain in the season of 1926-27, the water level in July was lower than it was for the preceding year. There is no synchronism between the fluctuations of Well No. L8N and any others listed. The lack of frequency of measurements for Wells Nos. 131 and 14, which have the least depth to water, prevents any accurate study being made as to the effect of rainfall. These might be pressure fluctuations from distant sources, as both wells are near the obstruction which creates the springs and artesian wells mentioned by the author.§ If, as the author states, "there is corroborative testimony that the flow of the wells and springs varies little over a period of years, the inference being that the comparatively compact formation acts as a regulator of an otherwise erratic recharge," and there is no transmission of pressure, it is strange that Wells Nos. 131 and 14 should show a maximum rise of 1.21 ft. and 1.10 ft., respectively, when the maximum rises for Wells Nos. 129 and L8N were only 0.26 ft. and 0.29 ft., respectively. If these fluctuations were due to penetration of rainfall, then it must be assumed that two of the wells showed four times the penetration of the other two.

*Determination of Rainfall Penetration from Test Wells in Pauba Valley on Temecula River, Riverside County, California.*—The line of test wells used by the author in his determinations in the Pauba Valley extend from the mouth of an arroyo draining a part of the North Mesa to the Temecula River,

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1147.
† *Loc. cit.*, pp. 1145, 1150.
‡ *Loc. cit.*, p. 1148.
§ *Loc. cit.*, p. 1151.

as shown on Fig. 4. The author states* that the storm of February, 1927, produced a rainfall of 14.65 in. in the Pauba Valley and, in Table 6, the same storm produced a rainfall of 9.14 in. at the test plot on the North Mesa. A rain of this intensity would certainly produce some run-off down the arroyo at the line of the test wells and add to the water supply. The author states that† "in some instances the measurements indicated a rise due to run-off from adjoining mesa land. On the other hand there was a compensating run-off away from bottom-lands." The writer can not comprehend how run-off from the bottom-lands would compensate for water which would be absorbed in the valley from the run-off from the mesa lands. The Temecula River was undoubtedly in flood during the heavy rainfall period in February, 1927, and passing as it does, through the pervious valley fill of the Pauba Valley, would also contribute to the rise in the wells. The writer believes that the data presented by the author are far from conclusive as to the amount of contribution to the underground-water supply by deep penetration of rainfall on the valley floor.

This paper contains material some of which has been used in litigation proceedings.‡ From this and other sources the writer is led to question some of the points mentioned in the paper that should be recorded.

Fig. 4 does not show correctly the drainage of Long Valley.§ Test Plot No. 1 is located about 1½ miles too far south; the location shown is in a group of rough, broken hills. The borings in the North Mesa Test Plot were made in duplicate and the samples averaged. Those portions of the samples not removed for office determinations were run back into the holes and tamped. The soil encountered in boring the holes ranged from a firm, semi-impervious soil to hardpan. The existence of the hardpan from 4 to 5 ft. below the surface shows the extreme depth to which rainfall has ever penetrated in this region. The tamping of the hardpan cuttings in refilling the test holes, a 2-min. operation, would leave a duct through which the ponded water in the test plot could percolate freely in comparison with the percolation through the undisturbed hardpan. The ponded area would prevent run-off escape. From Table 5 it appears that there were eight of these locally filled holes prior to the final determination on March 7, which would explain the presence of any excess moisture below 5 ft. under the test plot.

There is no general water-table underlying the mesas. The uncontradicted testimony in the case showed that suspended water-planes are found throughout the mesas, not by infiltration of rainfall on the surface, but by infiltration from water concentrating in gullies which cut the plain to contact with somewhat pervious lenses formed at intervals in the mesa silt.

The four wells submitted in Table 7‖ do not show true conditions. Near Well No. 129 is another one called Well No. 133, in which the water is usually from 65 to 75 ft. higher than in Well No. 129; near Well No. L8N is another

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1163.

† *Loc. cit.*, p. 1164.

‡ Rancho Santa Margarita *vs.* Vail, *et al.*, Superior Court of San Diego (Calif.) County, Case No. 42850.

§ See the Elsinore Quadrangle of the U. S. Geological Survey.

‖ *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1151.

called Well E7N in which the water level is out of harmony with the one selected by the author and which shows a reverse slope from the river bed; near Well No. 131 is another one known as the "old Shrode Well", which has a water-plane from 40 to 50 ft. higher than Well No. 131, although down the valley where it should be lower, if in the same water-table; and near Well No. 14 are several wells and springs on the August Cantarini Ranch, which differ from it in water-plane elevation. All these facts, shown by sworn and uncontradicted testimony in Court, prove conclusively that there is no water-table under the North Mesa region and no penetration of rainfall to a water-table, but that suspended water-planes are found locally in some semi-porous lenses in the mesa silt where such lenses are cut into and fed by run-off water gathered in gullies.

Well No. L8N shows a drop by March 16, which is positive evidence that the slightest rise of February 23 was caused by rain water running down outside the 2-in. pipe in this well. The well had been drilled by a rotary rig to a diameter of 5 in. in January, 1927, without casing, and a 2-in. standard lap-welded pipe inserted, the space between the outside of the 2-in. pipe and the inside of the 5-in. drill hole having been filled with loose soil. Of course, a little water would penetrate this loose fill from the heavy rains just before February 23, which would disappear before March 16. Well No. 131 had a depression in the ground around the casing which was 0.8 ft. above the bottom of this depression. There was much drainage from run-off into this depression. This caused the water to run into the well over the top of the pipe, whenever it became more than 0.8 ft. deep around the well, which did occur during the February, 1927, heavy rainfall. Further than this, the driller had tried out his perforating tool and had cut a number of slits in the casing just below the surface of the depression around the well, through which water was seen to enter during the February, 1927, rains.

The author assumes* that the flow of the springs and wells is entirely derived from deep penetration of rainfall on the mesas. He ignores entirely the fact that there is a drainage area of more than 100 sq. miles of low granite mountains, foot-hills, and plains within the drainages of Santa Gertrudes Creek, Cantarini Wash, and Shrode Wash, which all converge at the natural dam formed by the Elsinore Active Fault. In each of these drainages are sand and gravel fans deposited as modern alluvium within erosions into the Pleistocene "mesa silt" extending back from the fault line. The floods from this large back-country drainage flow over these porous fans and are absorbed in part, after which they percolate toward the Elsinore Active Fault which forces at least a part of the water up to the surface as springs.

The situation in Pauba Valley is a familiar one to those acquainted with artesian basins of the interior California valleys. There is a deeply eroded trough in the mesa silt, which is refilled to a depth of more than 500 ft. with modern alluvium. This lies between the "North Mesa" and the "South Mesa", as shown in Fig. 4. Its lower end is closed by the Elsinore uplifted fault block and into its upper end debouches the Temecula River, draining a high, mountainous area of 326 sq. miles. This stream first eroded the mesa silt

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1151.

and then refilled the space with porous recent alluvium. It left a coarse fan or delta at the upper end and finer deposits farther down the Pauba Valley. The flood waters of the river spread over and are absorbed in part in the fan which is called the outwash area, and rise through artesian pressure in the lower valley, which is called the artesian area. When it rains on the mountain water-shed and the resulting flow sinks and refills the outwash area, hydraulic pressure, transmitted through the saturated inclined valley fill, causes the wells in the artesian area to rise. It was this rise which the author mistook for direct penetration of rainfall.

The author cites the record of only three test holes in the Pauba Valley. There were others introduced in evidence, which, after floods in the Temecula River, showed a greater rise than would have been accounted for, if the entire quantity of rain had penetrated to the water-table.

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# PRE-DETERMINING THE EXTENT OF A SEWAGE FIELD IN SEA WATER

## Discussion *

By Harrison P. Eddy, M. Am. Soc. C. E.

Harrison P. Eddy,† M. Am. Soc. C. E. (by letter).‡—It seems rather unfortunate that the authors chose the conditions surrounding the Deer Island Sewer Outlet in Boston Harbor to assist in corroborating their theory on predetermining the extent of a sewage field in sea water;§ they were evidently misinformed as to the design features of the outfall itself. The data to which they have applied their theory are in error in two respects: The depth of outlet, which they call 30 ft., and the size of outlet, which they call 18 by 30 in.

The sewage is discharged from fourteen outlet openings increasing in size toward the outer end.‖ The diameters of these openings, which are approximately elliptical (with the exception of the outermost, which is a 48-in. circle), vary from 25 by 44 in. to 13 by 23 in., respectively. The mean area of the fourteen openings is, therefore, 640 sq. in., instead of 540 sq. in., as assumed by the authors.

From plans of the Metropolitan Water and Sewerage Board it is found that six of the outlets are at a depth of about 50 ft. below mean low water, while the other eight openings range in depth from 30 to 46 ft., with an average depth of 38 ft. Thus, the mean depth of the fourteen outlets is 43 ft. below mean low water, instead of 30 ft.

Had the authors used the correct values for depth and area of outlet, their statements§ in regard to the application of the formulas to the Deer Island Outfall would probably have read as follows: The length of path should be

* Discussion of the paper by A. M. Rawn and H. K. Palmer, Members, Am. Soc. C. E. continued from October, 1929, *Proceedings*.

† Cons. Engr. (Metcalf & Eddy), Boston, Mass.

‡ Received by the Secretary, October 15, 1929.

§ *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1187.

‖ "The Metropolitan Sewerage Works," by R. W. Loud, *Journal*, Boston Soc. of Civ. Engrs., October 8, 1923, p. 325.

52 ft., and $S_0 = 42$, theoretically. For $x = 300$, $S = 262$; and for $S = 212$, $x = 195$, and the field area, 2.7 acres, or 0.025 acres per sec-ft. If the observed value ($S_0 = 21$) of the initial dilution is used, the field limit at $S = 212$ is found to be at $x = 783$ ft. and the area, 44 acres.

These results differ radically from those which were obtained by the authors and which were deemed to corroborate their theory. The initial dilution obtained by applying the formulas to the corrected data is twice that of the observed initial dilution, and the equations, therefore, are decidedly not on the safe side in this instance. However, it may well be that the authors' Equations (1) to (35) are not applicable to such a complicated outfall, in which the outlet openings are of various sizes and are located at different depths.

In order to compare the results obtained by observation with those to be expected from an application of the authors' theory, a study has been made of some tests at the Nut Island Sewer Outlet in Boston Harbor.* This outfall consists of two 60-in. pipes discharging vertically, but at the time of the tests only one outlet was in use.

In order to determine the degree of dilution that takes place and the condition of the sea water before the sewage reaches the surface, several samples of water were taken immediately over the outlet in use on October 14, 1915. Samples were collected at the same time in the line in which sewage was flowing from the outlet and at various distances therefrom, as well as samples of harbor water from points outside the path of sewage from the outlet. The tests were made when the rate of discharge was approximately 1 380 000 gal. per hour, and when the depth of water over the outlet was about 30 ft.

Observed dilutions have been derived from the free ammonia content of samples collected at various points. For example, the initial dilution ($S_0$) is obtained from the free ammonia content of the sewage (17.600 parts per million), the surface of the harbor above the outlet (1.753 parts per million), and the harbor water outside the path of sewage from the outlet (0.120 parts per million). From these figures, $S_0 = 11$. In a similar way, the dilution near the outer limit of the sewage field, at a point 1 400 ft. from the outlet, where the free ammonia content of the water was 0.330 parts per million, has been found to be 83.

The velocity of the ocean current has been estimated from the time (1.37 hours) required for floats to travel from a point 1 400 ft. from the outlet to 3 500 ft., because the limit of the sewage field, as determined by observation and analysis, seemed to be approximately 1 400 ft. from the outlet. Within the sewage field, floats of course would be influenced by the dispersion of the sewage over the harbor surface.

Table 9 gives a comparison of the results obtained by computation, using Equations (1) to (35), with those obtained by chemical analysis and observation at the Nut Island Outfall.

In the case of the Nut Island Outfall the authors' formulas yield results well on the side of safety, except for $t$. In the latter instance the computed value checks fairly closely with the observed value.

* Rept. of the Mass. Dept. of Health, 1915, p. 347.

TABLE 9.—COMPARISON OF COMPUTED AND OBSERVED VALUES.

| Factors. | Computed value. | Observed value. |
|---|---|---|
| Initial dilution. $S_0$ | 5 | 11 |
| Time of travel ($t$), in hours, from head of rising column to edge of field. | 2.0 | 1.5 |
| Distance ($e$) that sewage will travel with current, in feet | 4 810 | 1 400 |

Another comparison between results obtained by observation and by the use of the authors' formulas is afforded by tests made by the State Department of Health at the sewer outlet of New Bedford, Mass., on August 9 and September 24, 1915.* The New Bedford Outfall,† which has a diameter of 60 in., discharges vertically through a single outlet at a depth of 30 ft. below low-water mark. The quantity of sewage flowing through the outlet at the time of the tests is not definitely known, but a flow of 100 gal. per capita per day has been assumed for the total population of the city in 1915 (109 568). The observed initial dilution has again been derived from free ammonia content, the determinations used, being as follows:

| | Free ammonia, in parts per million. |
|---|---|
| Sewage | 18.202 |
| Sea water above outfall: | |
| August 9, 1915 | 1.276 |
| September 24, 1915 | 0.207 |
| Sea water outside area affected by sewage: | |
| August 9, 1915 | 0.110 |
| September 24, 1915 | 0.044 |

When applied to the data given for the New Bedford Outfall, Equations (1) to (35) yield a value of $S_0 = 9$, while the values of $S_0$ derived from free ammonia content are 15 for August 9 and 111 for September 24. Here, again, the result obtained by the formulas is well on the side of safety. In fact, if the conditions found at the New Bedford Outlet on September 24 are at all usual, the authors' formulas would have been practically useless. On the other hand, it will be observed from the two tests cited that there is a wide variation in the value of the initial dilution above the New Bedford Outlet at different times, and many similar tests would have to be made before definite conclusions could be reached as to a fair average value of the observed initial dilution.

From the foregoing it is evident that caution must be exercised in using formulas for predetermining the extent of a sewage field in sea water, although they may well be used as guides to judgment.

* Rept., Mass. Dept. of Health, 1915.

† *Engineering News*, July 31, 1913, p. 216.

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PAPERS AND DISCUSSIONS

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# GOVERNMENT RECLAMATION POLICY IN BRITISH INDIA

## Discussion *

By E. R. Foy, Esq.

E. R. Foy,† Esq. (by letter).‡—The author gives a clear and interesting exposition of "colonization" as carried out by Government agency in British India. The problem of installing settlers on new lands in that country is shown to be a State function. The chief interest of this paper, therefore, lies in comparing methods used in India with those in the United States.§

In India, the control of all water rights is vested in the Supreme Government. Due regard, however, is being paid to the rights of the Native States which are governed by Indian Princes. If there is any clash of interests the matter is adjusted by arbitration. The projects for supplying irrigation water, either to old proprietary lands, or to new areas that are being colonized, are financed by the Government. An adequate income is obtained by means of water rates levied on each crop-acre irrigated, and by the sale of new lands to settlers. These projects are generally remunerative, but there are a few areas in which canals are "non-productive" and in such places irrigation projects are undertaken as protective measures to guard against periodic famines. In years of good rainfall, irrigation water is not needed, but there are years in which failure of the seasonal monsoon rains causes failure of all husbandry operations, thereby inducing heavy demands for canal water and thus increasing the income of these otherwise unfavorable projects. It must be realized, however, that no colonization programs are undertaken in areas where cultivation with natural rainfall is possible. Protective works are only provided for occupied or proprietary villages subject to periodic famines.

* This discussion (of the paper by E. S. Lindley, M. Am. Soc. C. E., presented at the meeting of the Irrigation Division, San Diego, Calif., October 4, 1928, and published in August, 1929, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., Ilchester, England.

‡ Received by the Secretary, September 28, 1929.

§ *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1193.

In America, it appears* that at first there was no settled policy and the earlier irrigation projects were the result of private enterprise. Some of these were for the mutual benefit of the local landholders and settlers, and others were purely speculative ventures. In some cases speculators disappeared as soon as they "had made their pile," leaving those unfortunates who had invested their money to carry on as best they could. Now, however, the Federal Government seems to be assuming complete control; but even on State irrigation projects the procedure does not appear uniform, and varying conditions prevail. Some settlers are required to pay full charges and costs, while others have special privileges as subsidies, remission of interest dues, etc. Assumption of complete control by the State, however, is a forward step and no doubt a uniform policy will be developed in due time.

When the writer was, in 1897, first posted to the Lower Chenab Canal Colony in the Punjab, "colonization" methods were just emerging from the experimental stage. Water was then plentiful, while land was available on absurdly liberal terms; but the difficulty lay in inducing settlers to break ground under adverse conditions in new surroundings, even when it held promises of wealth as an alternative to starvation in their old homes. After some years of slow progress, the Administration was just beginning to gain the confidence of the settlers. Since that time two other projects involving several millions of acres have also been completed and are now fully matured. There is now (1929) a fourth irrigation project being brought to completion, and there is a wild scramble for land at any cost. In the Sutlej Valley Project, 9 000 000 acres of new and old lands are being developed to expand irrigation in British India and in the two Indian States of Bahawalpore and Bikaneer. Water was turned into one of the series of these canals on this project in 1926. Ultimately, this part of the project alone will develop 1 000 000 acres of new land in British India, and more than that in the State of Bahawalpore. It will also insure an adequate supply to a large area of old village lands in both territories. The settlers of to-day find it hard to understand the hesitancy of their predecessors thirty-five years ago in accepting the first farm lands offered on these projects.

Mr. Lindley refers† to the political side of the question concerning high *versus* low rates. Fortunately, irrigation is not what is known as a "transferred" subject under India's "reform" administration. The ultimate decision on matters of policy rests with the Government and not with the Legislative Council; therefore, the danger of interference by political parties is not yet present.

Apparently, political influence is a factor in America. An eminent member of the U. S. Reclamation Service once made a short tour through the Punjab. In the course of conversation with the Governor of the Province the visitor said, "Keep irrigation out of politics. Do not give the farmers a vote." Coming as this did from a prominent citizen of democratic America, the Governor was rather surprised. The visitor then went on to explain that some rather good and financially sound irrigation projects in America had been

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, pp. 1193 and 1207.
† *Loc. cit.*, August, 1929, Papers and Discussions, p. 1563.

either wrecked or quite severely hampered by candidates for election to Congress, who secured the settler's vote by promising either cancellation of the unpaid balance or postponement of payment if elected. This condition is verified by Mr. Lippincott,* although he further explains that the State is beginning to adopt a more rigid attitude toward those who fail to pay.

An important detail not mentioned by Mr. Lindley is the absolute necessity of providing an ample communication system on irrigation projects before the arrival of the first settler. Of primary importance is the construction of a railroad. Next, all the main arterial roads and highways; and, third, the "feeder" roads to connect the main highways, the railway stations, and the near-by markets. These obvious conveniences were neglected forty years ago when the Lower Chenab Colony was first being developed, and no doubt that factor contributed greatly to the lack of enthusiasm for the new lands. In the latest of the developments (the Sutlej Valley Project) an existing railway was re-aligned so as to pass through the middle of the new colony, while a complete and carefully designed system of roads and main highways is being provided, and special marketing towns have been located at suitable points along the railroad. In these towns the competition to secure building sites is particularly active and splendid prices are being realized. The extra cost of these conveniences is being recouped by the higher prices that the settlers willingly pay for the service, because they are no longer harassed by the difficulties of produce transportation, and isolation from their old homes.

The writer is fully in accord with the lessons with which Mr. Lindley† concludes his paper.

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* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1193.

† *Loc. cit.*, August, 1929, Papers and Discussions, pp. 1563–1564.

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# THE PLANNING OF CAPITAL CITIES: DENVER, COLORADO

## Discussion *

By William T. Lyle, M. Am. Soc. C. E.

William T. Lyle,† M. Am. Soc. C. E. (by letter).‡—The selection of "The Queen of the Plains" as the first of American capital cities to receive the attention of the Society is an appropriate one. Denver stands unique in many respects and should embody in external characteristics the civic consciousness of an extensive region.

The engineer in his conception of city planning is apt to drift into materialism. In the design of capital cities, even more than in others, the planner must avoid an emphasis which might be characterized as materialistic. Of course, the city must be a comfortable and healthy place in which to live, with every consideration given to those matters which facilitate trade, promote industry, and simplify transportation; but it must be much more than this. It must be a concrete expression of noble civic ideals.

Denver has unquestionably sensed several of these ideals and is bringing them out in the new city plan. The capital of Colorado is fortunately situated, as those who have seen the Rockies from the Capitol Building and have viewed from Inspiration Point, a glorious mountain panorama extending from Long's Peak in the north to Pike's Peak in the distant south, can testify. In addition to parks, boulevards, and a civic center within the city limits, advantage has been taken of splendid opportunities for the acquisition and development of mountain parks. In her civic center the majesty of the law and the universal dispensation of justice are held up to view.

There are other ideas and ideals that should be emphasized, some or all of which may be in the mind of the planners. Denver eventually will become an historic center and should perpetuate in visible expression the early life of

* Discussion of the paper by S. R. DeBoer, Esq., continued from November, 1929, *Proceedings*.

† Prof. of Civ. Eng., Washington and Lee Univ., Lexington, Va.

‡ Received by the Secretary, October 10, 1929.

what was at one time a frontier, and, later, became a great mining center in the rush for gold.

The writer would suggest an emphasis on humanitarianism, charity, and morality without which civilization cannot survive. It would seem proper, especially in a city which has featured moral reform among the young, to bring out the idea of civic righteousness. Instead of zoning eleemosynary institutions practically off the map, they should be featured properly in the city plan. In medieval times, before the separation of Church and State, the Dom was featured prominently and was often the center of an attractive "street picture". To-day, the location of the city's churches should receive due consideration by the city planner in order to emphasize the religious idea in the life of the American people.

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## NIAGARA POWER

### Discussion *

By Messrs. R. D. Johnson and Glen E. Edgerton.

R. D. Johnson,† Esq. (by letter).‡—This paper is very forceful, logical, and instructive to those who are interested in hydraulic power and this, to-day, includes a large part of the population.

Mr. Gibson has touched upon the direct benefit accruing to a large population and the indirect benefit to the whole country through the distribution of Niagara power, which is sold much cheaper than the average, in other territories less favorably situated. He has effectively disposed of a misconception that power development at Niagara Falls has affected the level of Lake Erie. He makes clear the difficulty, if not the futility, of attempting to regulate the lake level either to diminish its fluctuations or to improve the flow of the Niagara River, but he does point out the great desirability of modifying the discharge over the Horseshoe Falls and increasing the flow over the American Falls, both as an assurance of the preservation of scenic values and as a means of releasing additional water for power purposes. A very important point in his recommendation is the fact that this work may progress gradually, testing each phase of the situation by measurements and observation, so that there is not the slightest danger of having to depend solely upon pre-determined engineering computations for the ultimate result.

Mr. Gibson's values of revenue received from the sale of Niagara power brings to mind immediately a popular misconception as to the intrinsic value of undeveloped water power in the minds of many, although it is believed their number is decreasing. A notion seems to have grown up that water power costs very little because Nature so easily furnishes the water as compared, for example, to the arduous mining and shipping of coal. It is this naive idea, perhaps, that has been largely responsible for a great deal of Socialistic propaganda that the natural water power belongs to the people and that private

* Discussion of the paper by Norman R. Gibson, M. Am. Soc. C. E., continued from November, 1929, *Proceedings*.

† Hydr. Engr., New York, N. Y.

‡ Received by the Secretary, October 8, 1929.

interests should at least be heavily taxed by the Commonwealth for the privilege of developing and distributing such power. A water power site, in general, only becomes valuable and able to compete with other kinds of power when: (1) The topography favors development; (2) the flow is dependable; (3) the very best of engineering talent is brought into play; and (4) the location is not too far from the power market.

Niagara power has the benefit, to a marked degree, of these four requisites, but it should not be forgotten that its great success to-day is due to the highest quality of engineering skill.

While it is true that, when a favorable water-power site has been skillfully developed in the vicinity of a proper market, it may deliver cheap power, yet it is quite as true that even a favorable site may cost so much to develop, because of ineffective design and blunders, to say nothing of the great risk always involved in corralling and handling large volumes of water, that the perpetual interest charge will run the cost above that of other kinds of power. It is clear to those well versed in this subject that private enterprise is just as proper and effective for the public good in water-power development as in any other branch of public utilities. Even what might be regarded as simple extension of the present Niagara power development could easily be almost ruined from an economic standpoint if not undertaken by those who have spent their lives at this business and in that locality.

To sum up, then, it should be understood that the great economic benefit to the whole population of the further development of the remaining available water-power resources is only possible to the fullest extent when carried out with the strictest competitive economy such as is well known to result from private enterprise. That the benefit of such economy is necessarily passed on, either directly or indirectly, to all the people is an important point of Mr. Gibson's paper.

GLEN E. EDGERTON,* M. AM. SOC. C. E. (by letter).†—The author has pointed out clearly the prominent place that the Niagara Falls power plants have taken in the development of the art of hydro-electric power generation. The utilization of this natural resource is not yet complete, and its problems will continue to occupy a conspicuous position in the future as they have in the past. Great credit is due the individuals who have contributed to the solution of these problems. Although such credit may not have been fully accorded, it is significant that many of the most prominent engineers with whom the writer has come in contact, in connection with water power development throughout the United States, have been associated at some time in their careers with the Niagara Falls development.

The pre-eminence of Niagara Falls power in the water-power industry is paralleled by its prominence in the affairs of the Federal Power Commission. The first license issued under the Federal Water Power Act of June 10, 1920, covered the existing plants on the American side. The application for a license was submitted originally on July 1, 1920, before the Commission was fully

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† Received by the Secretary, October 25, 1929.

organized for the transaction of business. Nine other applications for power development involving the Niagara River were submitted to the Commission, which gave them final consideration at a meeting of February 28, 1921, and authorized a license to be issued to The Niagara Falls Power Company for the then existing plants and the enlargment of the Schoellkopf Station to its present capacity.

The deliberations of the Commission in these cases involved some important features of the new water policy. Among them was the practical application of Section 10 (*a*), of the Federal Water Power Act, which reads as follows:

"Sec. 10. That all licenses issued under this act shall be on the following conditions:

"(*a*) That the project adopted, including the maps, plans, and specifications, shall be such as in the judgment of the commission will be best adapted to a comprehensive scheme of improvement and utilization for the purposes of navigation, of water-power development, and of other beneficial public uses; and if necessary in order to secure such scheme the commission shall have authority to require the modification of any project and of the plans and specifications of the project works before approval."

Before authorizing any project, it is incumbent on the Commission to determine whether it is "best adapted to a comprehensive scheme of improvement and utilization." While The Niagara Falls Power Company did present alternative schemes, including several types of over-all development, the application dealt primarily with the 215 ft. of head obtainable by diversion around the Falls only. Manifestly, this project did not of itself constitute a comprehensive development for the Niagara River to the extent of permissible diversions of water, because it left undeveloped the fall in the Lower Rapids, amounting to nearly 100 ft. It had already been established that a large quantity of water could be diverted around these rapids without injury.

The probable feasibility of over-all diversion was indicated not only by the alternative proposals of The Niagara Falls Power Company, but by two other applicants for authority to develop the total fall. The Commission, then, had at its disposal 20 000 cu. ft. per sec. of water which it could authorize for use in: (1) A development through 215 ft. of fall around the Falls, leaving the remaining fall for separate development; (2) a single development through about 310 ft.; and (3) a development using part of the water around the Falls only, with an over-all development for the remainder.

Investigation showed that the cost of development around the Falls alone gave the cheapest power. Capital costs per horse power of over-all development in a single stage would exceed those of a Falls development by about 40 per cent. Similar costs of the second stage, in a two-stage development, and the cost of the entire development in the two stages, would exceed the same figure by about 250% and 125%, respectively. Consequently, authorization of the Falls development would produce much the cheapest power immediately, but it would commit the Commission to an eventual scheme of development much more expensive than the best development, unless the latter should be postponed until the power plants at the Falls could be scrapped.

It appeared that the partial development proposed was capable of meeting the power requirements of the tributary region for several years; that

the more expensive development, although economically feasible, would necessarily involve higher charges for power to the disadvantage of industries which, in some cases, owed their existence to the possibility of procuring power at minimum rates; that additional power was needed urgently and could be supplied most promptly by a partial development; that probably additional diversions could soon be authorized in connection with remedial works at the Falls so as to permit a second step in development which might embrace the entire 310 ft.; that such a step would serve the urgent needs of the power market for an extended period; and that even in the ultimate development it will probably be possible to permit greater diversion around the Falls than around the Lower Rapids on account of the requirements of the ice-carrying capacity in the latter. The relative degrees to which these and similar factors governed the final decision of the Commission can not be stated precisely, but the conclusion was reached that the Falls development proposed in the Schoellkopf Station was best adapted to a comprehensive scheme of improvement.

A license was issued on March 2, 1921, to permit the use of 19 500 cu. ft. per sec. of water, the remainder being reserved to satisfy equitable rights that seemed to be vested in certain power users along Eighteen-Mile Creek below Lockport, N. Y., by reason of long use. Of this water, 225 cu. ft. per sec., which was subsequently determined to be in excess of the quantity required to satisfy those rights, was allotted to The Niagara Falls Power Company on December 13, 1926; and since rights themselves have now been acquired in the interest of that Company, it is expected that the entire quantity authorized by the Boundary Waters Treaty will shortly be available for the Schoellkopf Station.

This case illustrates the necessity for broad and reasonable interpretation of one section (and the same necessity applies to other sections) of the Federal Water Power Act. Little criticism of the principal conclusion is possible, but it might have been held that complete physical development in the most economical way should be required immediately. The results of such a finding would have violated the economics of the situation as it existed in 1921. The reservation of a quantity of water for relatively wasteful use at Lockport and along Eighteen-Mile Creek can not be so easily established in accordance with the principles of the law.

A limitation of the maximum rate of diversion to 120% of the average rate was fixed by the Commission in its license for the American plants. The pertinent words of the treaty are, "not exceeding in the aggregate a daily diversion at the rate of twenty thousand cubic feet of water per second." If it was intended to fix the maximum rate at 20 000 cu. ft. per sec., there seems to be no significance to the word, "daily." If it was desired to fix only the quantity to be diverted in each day, it might readily have been stated definitely in acre-feet or other convenient units. There is room for disagreement as to the exact force of the phrase, but the Commission, concluding that a higher rate than 20 000 cu. ft. per sec. was permissible, also believed that the public interest required a limit which would not permit diversion at a very high rate for a few hours each day. No such specific limitation is known to govern

the plants on the Canadian side, which, the writer believes, actually operate with a peak somewhat higher than 20% above the average rate of diversion.

The Federal Power Commission undertakes to preserve natural scenic grandeur and beauty wherever they are of importance in connection with any prospective license. This responsibility is considered to be a part of the obligation, laid down in the section of the Federal Water Power Act previously quoted, to insure provision for other beneficial public uses in addition to power development and navigation. Such questions have been prominent in many water-power projects beside those at Niagara Falls. Developments on Lake Chelan, in Washington; Wolf River, in Wisconsin; Cumberland River, at Cumberland Falls, in Kentucky; Arkansas River, at Royal Gorge, in Colorado; and the Potomac River, at Great Falls, in Maryland and Virginia, are conspicuous cases in which strenuous objections to the issuance of a license have been made by organizations and individuals, concerned lest the Commission might fail to recognize or to perform effectively its duty in such matters.

In the Niagara Falls case it was not considered that further studies, in addition to those which had resulted in the treaty provisions, were necessary to establish the desirability of permitting the diversions approved therein, but an article was included in the license to require the power stations to be constructed and maintained so as to blend harmoniously with the scenic features near them.

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## FILTERING MATERIALS FOR WATER AND SEWAGE WORKS

### PROGRESS REPORT OF THE COMMITTEE OF THE SANITARY ENGINEERING DIVISION

### Discussion*

By J. E. Lamar, Esq.

J. E. Lamar,† Esq. (by letter).‡—This report considers the disintegration of filter stone from two standpoints: (1) Mechanical weathering; and (2) chemical weathering, although these two divisions are not expressly stated. Under the first class is the action of freezing and thawing and the effects of temperature changes. The blowing up of chert, which is mentioned in several places in the report, takes place also in the case of chert used as aggregate for the construction of concrete roads. The pieces that are near the surface of the road blow up or "pop". These pieces are commonly porous, chalky chert, and this condition has apparently been developed by chemical weathering. The actual "popping" of the chert, however, may probably be attributed to mechanical weathering.

Chemical weathering, that is principally solution, works hand in hand with mechanical weathering, the one accelerating the other. The solution of certain parts of filter materials when in service is without doubt a real phenomenon tending toward the destruction of these materials. The Committee mentions§ the presence of dissolved carbon dioxide in sewage. This gas is a very important agent in causing the solution of limestone and dolomite. As this is the case, it seems worth while to obtain, if possible, an average chemical analysis, indicating especially the amount of carbon dioxide present, of the

* Discussion of the Progress Report of the Committee of the Sanitary Engineering Division on Filtering Materials for Water and Sewage Works, continued from September, 1929, *Proceedings*.

† Geologist, Non-Fuel Products Section, State Geological Survey, Urbana, Ill.

‡ Received by the Secretary, September 27, 1929,

§ *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1805.

sewage being sprayed over any given filter bed, in order that the importance of solution with regard to the life of filter stone may be gauged more accurately.

Another line of investigation which might yield some important data would be a study of the relative frequency of freezing and thawing at the different plants described. Undoubtedly, the weather as affected by latitude and local conditions, and also the method by which the filter is operated during the winter months, will bear on the rate of mechanical weathering. In this connection it should be noted that the difference between the maximum and minimum yearly temperatures is less important than the number of fluctuations of temperature above and below freezing.

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## CHARGES AND METHOD OF MAKING CHARGES FOR PROFESSIONAL SERVICES

### REPORT OF SPECIAL COMMITTEE

### Discussion *

BY ORMOND A. STONE, M. AM. SOC. C. E.

ORMOND A. STONE,† M. AM. SOC. C. E. (by letter).‡—An authentic background for engineering charges, particularly when dealing with laymen, is of the greatest value to the engineer in private practice. The report of the Committee is earnestly recommended for careful study by every member of the Society as a step toward establishing the engineer in the minds of the general public as a member of a worthy profession.

The Committee is particularly to be commended for the excellent manner and thoroughness with which it handled an exceedingly complex and difficult problem. Primarily, the charge made for all commodities, labor, or service is a function of the cost of production. The merchant, manufacturer, or industry that neglects to give proper consideration to the importance of cost accounting and systematic records of the financial end of the business is doomed to early failure in this age of keen competition. Because of the variety of the work involved and the larger number of assistants and employees required, engineering presents a more difficult problem from the standpoint of cost accounting than, perhaps, any of the other professions.

The training of the young engineer is devoted almost wholly to the practice of his profession with very little opportunity offered to guide him in estimating the cost of producing the work for which he is employed. This statement is to be applied not to the cost of constructing the material things or improvements which he is employed to plan, but rather to the cost of producing the plans to which the structure or improvement is to be built.

* This discussion (of the Report of the Special Committee on Charges and Method of Making Charges for Professional Services, published in September, 1929, *Proceedings*) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Civ. Engr. (Olmsted & Gillelen), Los Angeles, Calif.

‡ Received by the Secretary, October 28, 1929.

Any step taken by the Society toward educating its members and the public to fees for engineering service commensurate with careful research and fully executed plans will unquestionably re-act favorably, both toward the profession and the public, by eliminating the carelessly drawn and incomplete plan, which necessitates expensive changes during construction, and by allowing more accurate and competent bidding by contractors on public work.

The question of bookkeeping to the average engineer in private practice is more or less abhorrent. Usually such books as are kept concern themselves only with disbursements and accounts of clients. Until such time as the professional engineer can be made to realize the actual cost to him of producing the work for which he has contracted his services, it will be extremely difficult to make many members of the profession realize that percentage fees may not safely be cut below those established by other members, whose accounting systems dictate the remuneration which should be received for a certain class of service.

The cost of producing engineering work cannot be known unless a logical and adequate method of cost accounting is adopted. It reasonably follows, therefore, that charges for service and cost accounting are intimately allied. The development of a cost-accounting system which is practical in its application and, at the same time, is in control with the general ledger accounts is an interesting problem and one which should appeal to the fancy of the man whose training has stressed the importance of logical analysis.

It is believed that in general the feeling exists among engineers that bookkeeping belongs in the realm of the accountant, and that accounting problems should properly be turned over to a member of this profession. Serious thought on the subject soon makes clear the fact that the accountant must be guided in making his set-up by the character of information desired and that only the man thoroughly familiar with the intricacies of the business is in a position to know what type of cost records will be most useful.

To illustrate that the problem is not as complicated as usually considered and may be made the source of accurate data within remarkably close limits, a few "high lights" on a method which has been in successful operation in several engineering offices in Los Angeles, Calif., for many years, are presented herewith.

Consider, first, the question of salary costs. Every one connected with the organization and the production of any part of the firm's business must, if proper cost records are desired, be put on a definite salary or wage scale. In the case of a partner or executive having an interest in the profits, a reasonable salary commensurate with the remuneration of men of similar ability in the employ of corporations, or of some branch of civil service, should be established. This salary may be termed a drawing account, or what you will, but it must be set up for the purpose of cost accounting.

The time spent by any member of the organization in producing work is, of course, charged to the different jobs as a direct salary cost and at a per diem rate. The per diem cost to be charged must be made the subject of some thought and any assumption that a man will average twenty-five working days per month will soon be found to be entirely erroneous.

If an employee is paid by the day or hour there is, of course, no question as to the amount to be charged to cover his direct salary cost. If he is paid a monthly salary, the problem is not quite so simple. Experience has proved that the average office assistant works not more than twenty productive days per month as a long-time average. His salary cost is then found by dividing his salary by twenty, which gives the cost per day to be charged to the job.

A junior member of the firm may spend only a part of his time on productive work, the remainder being required to investigate prospective business or on other work of an advertising or promotional character. The latter must be distinguished from time spent on jobs in progress as it is strictly an overhead cost and should be accounted for as such. It may be carried as an indirect salary cost, or promotion salary cost, and, being a part of the overhead, must not be confused with direct salary costs.

Any monthly variation shown on the time sheets between the salary charged to cost and the salary earned or paid is either charged or credited to a reserve account for salary fluctuations, which at the end of the year may be closed out into general overhead or carried over from year to year. Vacation salaries are charged to this reserve account; likewise, salaries or wages paid during illness.

Salary costs alone are only a part of the expense involved in producing engineering work. Overhead expenses are a reality admitted by all, but they are under-estimated or overlooked by many. The theory that a margin of profit sufficient to cover overhead may be added to the salary cost is more often than not apt to prove an expensive fallacy. The overhead expense properly chargeable to each job should be known and can be determined with a well-adjusted system of cost accounting. If overhead costs are analyzed it is early discovered that they naturally fall into two distinct groups.

The first group covers the general fixed charges pertaining to the business and includes such items as office rent, depreciation on equipment, clerical assistance, telephone and telegraph, taxes, advertising, office supplies, postage, etc., and will be found to be more or less stable from year to year. If such items for an office totaled $6 000 during 1929, it is fairly safe to assume that they will not be much more, or little less, for 1930.

The second group includes intangibles, and may fluctuate widely from year to year, dependent on the character of the business. Time devoted to the promotion of new accounts, civic duties, society affairs, or investigation and research not directly remunerative, are all parts of the overhead cost of the business. Time spent by assistants in filing, or in such other duties as may not be directly charged to an individual job, likewise must be classed as an overhead and falls into the second group.

The importance of accounting for this part of the overhead of any average sized engineering office can only be realized after a careful accounting, covering a year's time, has been studied. The time and the cost are apt to prove rather astounding. With the realization that overhead must be properly charged to cost, the question arises as to the best method to accomplish this objective. If it is true that the fixed charges remain fairly constant from year to year, it is comparatively simple to ascertain their approximate amount during the

year past and to estimate the monthly quota which must be charged to work in progress to cover this part of the overhead. To estimate the probable amount of the second group requires a full insight into the business of the office, the amount of work on hand or in prospect, and, most emphatically, may not be left to any accountant or subordinate to determine.

During the first year of carrying a set of cost accounts, it will be found that adjustments to the overhead charged to cost must be frequently made. Subsequent years will develop that this most intangible portion of business cost may be estimated with surprising accuracy. For the purpose of illustration, it will be assumed that for the same office, with fixed charges of approximately $6 000 per year, the indirect salary costs (second group) are estimated at $6 000 also. Thus, $12 000 per year, or $1 000 per month, must be distributed into cost to cover overhead expense. The most natural method of distributing this cost to the job is apparent and involves adding a surcharge to the salary cost of the men employed. This method has been used with perfect satisfaction for some years and requires little comment except that the following points should be borne in mind:

1.—The major portion of the overhead costs should be carried by the office force producing the work.

2.—Field forces may be more or less transient and, in general, should carry little or no overhead.

3.—Executives will probably not average more than 10 to 15 days per month directly chargeable to any particular job, and this must be considered in determining the overhead they should carry.

4.—It is understood that indirect time, being a part of the overhead cost, must not carry any overhead surcharge.

In practice, a time sheet is made out each month by every member of the organization, and itemized as to the work done on each job. The total time on each sheet chargeable to each account is multiplied by the direct salary charge and by the overhead surcharge, and these separate amounts are listed in the space provided on the time sheet by the bookkeeper.

Salary charged to cost and salary earned are also set down on the time sheet, and the proper charge or credit is made to the reserve account for salary fluctuations.

Entries to the cost account ledger are made directly from the time sheet. Entries to the general ledger are made from a summary sheet which is a condensed statement of the information shown on all the time sheets and, further, serves as a voucher for general ledger entries.

The cost ledger, clients' ledger, and general ledger are all combined in one binding. The cost ledger sheets are printed on the reverse side of the clients' ledger sheets so that the two are always readily comparable. The cost ledger sheet is divided into colmns headed "Travel Expense", "Salary Costs", "Overhead Costs", "Costs for Month", "Accrued Cost", "Earnings for Month", "Accrued Earnings", and "Accrued Balance."

Each month a voucher is made by an executive familiar with the status of the office business on which it set forth the charges for the month and the estimated earnings.

Monthly earnings are set up by estimating what percentage of the work has been accomplished during the month and has nothing whatever to do with the time or method of payment which may be stipulated in a contract. If one-half the work on a contract has been done, one-half the total fee has been earned, regardless of whether the job was paid for in advance or may not be billed until the work is completed.

In order that the monthly statement and the general ledger may show the true status of the business, estimated earnings and charges for service are handled through an account called reserve for advance earnings. All charges are credited to this account and charged to the clients' account. All earnings are charged to the reserve for advance earnings and credited to the revenue account.

By means of a cost control account the cost ledger is kept in balance with the general ledger. Other control accounts are utilized to minimize the work of keeping the ledgers in balance but, being largely a matter of accounting, will not be described.

While the system herein described may sound complicated and cumbersome, it has been found by several firms to be accurate, dependable, and, once understood, is handled by an intelligent secretary with two or three days' work per month.

As a balance sheet and statement is made each month, it is possible to compare the overhead charged to cost with the actual accrued overhead expenses, and thus permit adjustments to be made if they are required.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

# ESSENTIAL FACTS CONCERNING THE FAILURE OF THE ST. FRANCIS DAM

## REPORT OF COMMITTEE OF BOARD OF DIRECTION

## Discussion *

BY MESSRS. FRANK S. BAILEY AND JAMES R. GRIFFITH

FRANK S. BAILEY,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The Committee's report, including the abstracts from the reports of the six commissions appointed to investigate the failure of the St. Francis Dam, is of high value. It is to be hoped that the clear presentation of these findings will tend to lessen the danger of similar calamities in the future.

It is a great pity that a commission similar in personnel to any of these that were appointed in connection with the failure, was not engaged to consider the safety of the dam when the large cracks appeared and the seepage increased. If the weakness of the foundation material had been discovered by such a precautionary commission, the tragedy might possibly have been averted by opening all the outlet pipes and keeping the water in the reservoir well below Elevation 1835. If the outlet pipes were not of sufficient capacity to keep the reservoir at a safe level, and it was fully realized that a catastrophe impended unless more radical measures were utilized, it might have been feasible to devise such measures.

It seems obvious that if one or more large discharge tunnels, lined with steel and controlled by electrically-operated butterfly valves, had been built through the dam near the bottom it would have been possible to lower the reservoir in a comparatively short time. The situation could then have been studied and an attempt made to render the foundations safe by extensive grouting operations and the construction of deep cut-off walls and drainage

* This discussion (of the Report of the Committee of the Board of Direction on Essential Facts Concerning the Failure of the St. Francis Dam, published in October, 1929, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Structural Designer with New England Power Constr. Co., Boston, Mass.

‡ Received by the Secretary, October 15, 1929.

lines; or, if such measures were regarded as impracticable, the dam could have been demolished without damage to other property or loss of life.

Incidentally, it may be stated that, when much seepage appears at the base or sides of a dam, this water should be collected into ditches, and weirs or other measuring devices should be installed for the purpose of keeping a careful record of the variations in quantity. The cracks which appear in the masonry of a dam should be located accurately on a plan and measured at frequent intervals so that any change can be known almost as soon as it occurs. As a general rule all newly built dams should be kept under close observation. It is fortunate that a dam usually shows signs of weakness some time before its condition becomes really dangerous.

After a dam has failed, it is manifestly easier to explain why it failed than to predict positively, while it is still standing, that it is certain to fail. The writer knows of an earth dam that was decidedly leaky for two or three years after it was built. The reservoir would not fill up to the crest of the spillway because of excessive seepage; and yet this seepage finally diminished, owing probably to the deposition of silt near and on the up-stream slope of the dam, and the reservoir was filled without any harmful result.

Quite probably those who had charge of the St. Francis Dam expected some such favorable result. It is seldom that a site for a dam can be found which is ideal in all respects, and the need for an addition to a city's water supply is sometimes so urgent that a choice is made between various sites, none of which is exempt from unfavorable features.

Under such circumstances a site may be selected which the local authorities believe or hope can be made to answer their purpose, and in such case the value of a review of the project by a State supervisory commission is readily apparent. Such a commission would scrutinize the data presented by the local authorities and cause an examination of the site to be made by one of its own engineers before deciding to approve or reject the plans. Such a commission would have its own reputation for usefulness to maintain and would properly be expected to do much toward preventing the erection of unsafe structures.

Even such a regulatory body would not be expected to possess omniscient knowledge. It sometimes might condemn projects which would be safe and economically sound and approve some which later would prove disastrous; but, to modify a sentence* in the report of the Jury Drawn by the Coroner of Los Angeles County, "the checking by independent experts should reduce the effect of human error." It is not surprising that the State of California has recently enacted legislation with this intent.

JAMES R. GRIFFITH,† Assoc. M. AM. SOC. C. E. (by letter).‡—None of the reports on the failure of the St. Francis Dam mentions any possible weakening of the mica schist other than possible sliding due to lubrication of the planes of schistosity.

* *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2162.

† Prof. of Structural Eng., Civ. Eng. Dept., Oregon State Agri. Coll., Corvallis, Ore.

‡ Received by the Secretary, October 14, 1929.

The writer's experience has convinced him that at least sea water has a decided deteriorating effect on the structure of mica schist. Core drillings at the site of the Glen Island, New York, Bridge brought up only disintegrated material from the top strata. Large pieces of mica schist picked up along the seashore were soft enough on the surface to be easily disintegrated by a pen knife although the interior portions were still hard.

No reports as to the action of water on the strength of the mica schist under the east abutment of the St. Francis Dam are available. The writer has been very dubious of this material since seeing it first mentioned in reports of the failure. Very likely a decrease in compressive strength might be indicated, if tests were made of this schist under the action of water.

# MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

## CHARLES MACDONALD, Past-President, Am. Soc. C. E.*

DIED JULY 8, 1928.

Charles Macdonald was born at Gananoque, Ont., Canada, on January 26, 1837. He was the son of W. Stone Macdonald and the grandson of Charles Macdonald, who married the daughter of Colonel Joe Stone, the founder of Gananoque. After attending the public schools of his native town, and, later, a preparatory school of what was then Queen's College, Kingston, Ont., he obtained a position on the surveys of the Grand Trunk Railway between Kingston and Brockville, Ont., on which he served during 1852 and 1853. In 1854, Mr. Macdonald entered Rensselaer Polytechnic Institute, at Troy, N. Y., which was then the only institution of its kind in the United States, and was graduated therefrom with the degree of Civil Engineer in 1857. At the time of his death he was a Trustee of the Institute and its oldest graduate. In later years, he also received the degree of Doctor of Laws from Queen's University, Kingston, Ont.

Following his graduation Mr. Macdonald was engaged on the construction of an extension of the Grand Trunk Railway from Huron, to Detroit, Mich. Little engineering work was done in Canada during the next few years, although Mr. Macdonald lived at Gananoque until 1863. In the early spring of that year he moved to the United States, taking a position in charge of the Engineer Corps of the Philadelphia and Reading Railroad Company, with headquarters at Pottstown, Pa. Shortly afterward, he joined the Pennsylvania Volunteers, advancing to the rank of Corporal. He was made a prisoner in the Battle of Gettysburg.

Following the close of the Civil War Mr. Macdonald devoted much time to the study of iron bridge construction which was then in its infancy. A pamphlet on this subject which he wrote, and which found favorable attention, caused him to be entrusted with the construction of the bridges in the extension of the Delaware, Lackawanna and Western Railroad, to Hoboken, N. J.

In 1869 he opened an office in New York, N. Y., devoting himself to the design and construction of iron bridges. Mr. Macdonald was also much interested in the development of swing bridges. In 1871-1872 he designed and built the Point Street Bridge† in Providence, R. I., which was in continuous service under constant traffic until it was replaced by a heavy, wide, modern structure during 1927. He was appointed by the City of New York as a Trustee of the East River Bridge, of which Colonel Washington A. Roebling was Chief Engineer, which was opened in 1883.

* Memoir prepared by W. G. Triest, M. Am. Soc. C. E.

† Description appears in *Engineering*, Vol. XV (1873), p. 202.

In 1884, Mr. Macdonald organized the Union Bridge Company (of which he later became President), with headquarters in New York and shops at Athens, Pa. Of his partners, George S. Field, M. Am. Soc. C. E., and the late Edmund Hayes, M. Am. Soc. C. E., had their offices in Buffalo, N. Y.; and the late Charles Kellogg and Charles S. Maurice, Members, Am. Soc. C. E., in Athens; that of the late Thomas C. Clarke, M. Am. Soc. C. E., was in New York. The Union Bridge Company built many large bridges in the United States, including the Poughkeepsie Bridge, erected in 1887 and 1888. The Company also designed and built the Hawkesbury Bridge, in New South Wales, Australia, during 1886 and 1887. Both these structures had long spans and unusually deep foundations.

He was one of the earliest promoters of a suspension bridge over the Hudson River at 57th Street, New York. A large model of this bridge was on view in his office. He was active in the preliminary work and the organization of the American Bridge Company, to which he sold the Union Bridge Company in 1900; he served as one of the Vice-Presidents of the former company until the following year. He then retired from active business, but was occasionally engaged in consulting work, notably in 1910, as one of the Commissioners of the Quebec Bridge during its reconstruction.

Mr. Macdonald was very much interested in the growth and affairs of the Society, and contributed frequently to its *Transactions*.* He also discussed many papers presented by others, particularly those relating to bridge construction and materials. He was an early advocate of the use of steel in place of wrought iron in bridges and during 1879 proposed the use of high carbon steel for eye-bars and suggested the practicability of making steel eye-bars having a yield point of 63 000 and an ultimate tensile strength of 106 000 lb. per sq. in., which has been accomplished during the past few years.

Socially, he was much interested in the Engineers' Club, of which he was made an Honorary Member, the Century, the Lawyers', the University, and Union Clubs, of New York, and the Engineers' Club, of Montreal, Que., Canada. Mr. Macdonald was accustomed to divide his summers between his home at Rumson, N. J., where he was one of the first summer residents, and his place "Blinkbonnie", at Gananoque, on the shores of the St. Lawrence River.

In 1861, he was married to Sarah Louise Willard, the daughter of Colonel Willard, of Troy, N. Y. There were three children—a son, William Stone Macdonald, and two daughters, Mrs. John Serson and Mrs. W. G. Triest. After his wife's death in 1912, Mr. Macdonald made his home with his children, but he decided to return to his native town in Canada in 1926, where he spent his last years. He died there on July 8, 1928.

Mr. Macdonald was elected a Member of the American Society of Civil Engineers on September 15, 1869. He served as Director in 1871, and from 1874 to 1876; as Vice-President in 1893 and 1894; and as President in 1908.

* "Proportions of the Heads of Eye-Bars," *Transactions*, Am. Soc. C. E., Vol. II (1872-73), p. 333; "The Failure of the Ashtabula Bridge," *Transactions*, Am. Soc. C. E., Vol. VI (1877), p. 74; "The Ocean Pier at Coney Island," *Transactions*, Am. Soc. C. E., Vol. VIII (1879), p. 227; "The Six-Hundred Ton Testing Machine at the Works of the Union Bridge Company at Athens, Pa.," *Transactions*, Am. Soc. C. E., Vol. XVI (1887), p. 5; and "Presidential Address, at the 40th Annual Convention at Denver, Colorado, June 23d, 1908," *Transactions*, Am. Soc. C. E., Vol. LXI (1908), p. 544.

## WALDO CLAYTON BRIGGS, M. Am. Soc. C. E.*

DIED DECEMBER 13, 1926.

Waldo Clayton Briggs was born in New Haven, Conn., on March 14, 1870. He was the son of the late William Alexander Briggs, a merchant of New Haven, and Sarah Maria (Baldwin) Briggs, both of whom traced their ancestry back to the Colonial days of New England. His forebears fought on the patriot side in the American Revolution.

Mr. Briggs attended the public schools of New Haven and was graduated from the Hillhouse High School in 1889. The following three years he spent in the Sheffield Scientific School of Yale University, from which he was graduated in 1892 with the degree of Bachelor of Philosophy.

During his vacation periods, while he was in college, Mr. Briggs worked on surveying and drafting for the City Engineer of New Haven, and, in 1892, following his graduation, he continued in the same type of work with the City Engineer of Hartford, Conn. In 1893, he returned to New Haven as Resident Engineer on the construction of a masonry dam, about 1 000 ft. long, for the New Haven Water Company.

From 1894 to 1897, he served as City Engineer and Street Commissioner for South Norwalk, Conn. His next engagement, covering the period from 1898 to 1900, was as a Civilian Engineer with the Corps of Engineers, United States Army, on computing, drafting, and superintending the construction of fortifications at the eastern entrance to Long Island Sound. This connection was severed when the appropriation for the work was exhausted.

Mr. Briggs went to New York City in 1900 and, thereafter, until the time of his death, he was engaged on the construction of important engineering projects in and about the city. From October, 1900, to March, 1901, when he resigned, he was an Assistant Engineer on the First Division, Rapid Transit Railroad Commissioners, supervising the construction of a part of the first rapid transit subway. Subsequently, until 1904, he was employed with the Degnon-McLean Contracting Company, as Assistant Engineer, later as Engineer in Charge, under the Chief Engineer of the Company, on its contract for the building of that part of the first rapid transit subway which extended from 41st Street and Park Avenue, through 42d Street and Broadway, to 47th Street. At that time, subway construction was of the nature of pioneer work in New York City, and experience in working out the problems involved, by which those who built the later subways benefited, was then lacking. Those, like Mr. Briggs, who so successfully accomplished their difficult task under such conditions, deserve commendation for the ingenuity, foresight, and skill they displayed.

In 1905, when the Pennsylvania Railroad Company began its construction of the great project for a passenger terminal in New York City with all the appurtenant work, including tunnels under the Hudson and East Rivers, Mr. Briggs became the Resident Engineer for the New York Contracting and Trucking Company on its contract for excavating the site of the passenger

* Memoir prepared by Robert Ridgway, Past-President, Am. Soc. C. E.

terminal in the blocks bounded by 31st and 33d Streets and Seventh and Ninth Avenues, and for constructing the retaining walls and viaducts in connection with this project.

In 1907, he accepted a position with the Degnon Contracting Company, and, at first, was engaged on the preparation of bids and the supervision of various contracts. The following year he was made Chief Engineer of the Degnon Realty and Terminal Improvement Company, and was in charge of the contract for the construction of the large Sunnyside Yards of the Pennsylvania Railroad and of the real estate development in Long Island City, N. Y., continuing in this capacity to the end of 1912. The successful completion of the construction of the Sunnyside Yards, in low-lying marsh land, gave him much satisfaction.

From 1913 to 1916, Mr. Briggs was Chief Engineer for Carpenter and Boxley and Herrick on the construction of Sections 11-B-1 and 11-B-2 of the rapid transit subway from 40th Street to 86th Street, in Fourth Avenue, Brooklyn, N. Y., at a cost of $3 725 000. On the completion of this work, he returned to the Degnon Contracting Company, for the period, 1917 to 1920, as Engineer and Superintendent on two other Brooklyn subway contracts, one a deep tunnel in sand, 4 300 ft. long, known as Section 2-A of Route 12, the other, a subway station, directly under the existing Flatbush Avenue Terminal of the Long Island Railroad, known as Section 1-B of Route 12. These contracts involved a cost of about $2 000 000. Although Section 1-B was of relatively short length, the construction was so involved as to require careful planning and execution. Mr. Briggs brought both contracts to successful completion, after which he severed his connection with the Degnon Contracting Company, as that Company had decided to discontinue its contracting activities.

During 1922, the Thomas Crimmins Contracting Company of New York City employed Mr. Briggs as Superintendent on a $2 000 000 contract in Westchester County for installing additional steel pipes in certain siphons on the Catskill Aqueduct of the New York City Water Supply. From 1923 to the time of his death in 1926 he was employed by Henry Steers, Incorporated, of New York City, as Engineer and Superintendent. For the major part of these four years he was engaged on the difficult task of installing foundations for the large plant of the Western Electric Company at Kearney, N. J., on the marshy ground of the Passaic River meadows. The contract involved the expenditure of about $4 000 000, and, in the words of a member of Henry Steers, Incorporated:

"Not only by his engineering ability as shown by the efficient and able handling of this intricate work, but by his experience, his personality, and the many sterling qualities of his character, which we came to know, he established himself as a permanent and valued member of our staff. He acted in an advisory capacity to Mr. Steers on the various projects in which we were interested, and at the time of his death, he had an assured and brilliant future in this Company. By his untimely end, we feel deeply the loss of a valued business associate and a friend."

Death came to Mr. Briggs suddenly, without warning. On the morning of December 13, 1926, while leaving the City Hall of Jersey City, N. J., where

he had gone on business connected with plans for work in progress in that City, he had a heart attack and died almost instantly.

Waldo Briggs was a man of positive characteristics—one to whom right was right and wrong was wrong. He was intolerant of anything that was not direct and open and drove straight to the conclusion that, in his judgment, was the correct one. Absolutely loyal to a friend and a cause, he was indefatigable in his attention to any work he had in hand, studying its particular problems, never satisfied until he had mastered its details and thought out ways of overcoming its difficulties. His success was due, in large measure, to his straightforward and courageous way of looking at his problems and to his courage in acting as his judgment directed. One remembers long, and appreciates deeply, the friendship of such a man. He was of the army of constructive men who leave the world better than they found it.

Mr. Briggs enjoyed country life and out-of-doors sports. He was fond of dogs and on his long rambles, one, and sometimes two, were his companions. Biographies of noted men always interested him, and he read many, making copious notes of his impressions. In line with this, his interest in civic affairs was deep and he took the active part of a good citizen in the local affairs of his community.

He was married on September 2, 1897, to Belle Ferris, of South Norwalk, Conn., who, with a son, Ferris Briggs and a daughter, Amy Ferris Briggs, survives him. He was a member of the Scarsdale Golf Club, the Carteret Club of Jersey City, and the Yale and Quill Clubs of New York City.

Mr. Briggs was elected a Junior of the American Society of Civil Engineers on December 5, 1893, and a Member on July 10, 1907.

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## LOUIS HYDE EVANS, M. Am. Soc. C. E.*

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DIED JUNE 5, 1929.

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Louis Hyde Evans, the son of Enoch W. and Caroline Hyde Evans, was born at Kenosha, Wis., on January 21, 1851. He was the second oldest of a family of four children, two sons and two daughters. His father was a prominent attorney in Kenosha, and Chicago, Ill., to which city the family moved during the Sixties.

After preparation in the public schools of Chicago, Mr. Evans commenced his railroad work, in 1871, as Rodman, then as Instrumentman, with the Chicago and Northwestern Railway Company on the location and construction of its lines in the Northern Peninsula of Michigan. In the fall of 1872 he resigned to enter Rensselaer Polytechnic Institute, at Troy, N. Y., in the Class of 1876. He left the Institute, however, after six months, and entered

* Memoir prepared by a Joint Committee of the Society and the Western Society of Engineers, consisting of Lincoln Bush, Past-President, Am. Soc. C. E., E. C. Carter, M. Am. Soc. C. E., and J. S. Robinson, M. W. S. E.

the University of Michigan at Ann Harbor, Mich., as a special student in Engineering in the Class of 1875. In July, 1873, he obtained a leave of absence from the University to accept a position with the Erie Railroad Company as Assistant Engineer on the construction of double track on the Delaware Division of that road, grading and bridging 25 miles between Lackawaxen, Pa., and Calicoon, N. Y. He then returned to the University at Ann Arbor in the fall of 1874.

Following his graduation Mr. Evans was again employed with the Chicago and Northwestern Railway Company, and from June, 1876, to 1879, was Assistant Engineer on location and construction of its lines in Minnesota, on the completion of which, in 1879, he was employed on the Central Branch of the Union Pacific Railroad. In the latter part of 1879 he again returned to the Chicago and Northwestern Railway Company and served as Division Engineer and Engineer of Track Elevation in Chicago until 1900, when he resigned to accept the position of Assistant Chief Engineer with the Delaware, Lackawanna and Western Railroad Company at Hoboken, N. J. About 1904 he resigned that position to become a member of a firm of contractors who were making grade reductions on the Delaware, Lackawanna and Western Railroad in Northern New Jersey; after the completion of this work he returned to Chicago as Consulting Engineer for the Union Stock Yards and Transit Company, on its track elevation.

When this work was finished, Mr. Evans became associated with the late Horace G. Burt, Civil Engineer, formerly Chief Engineer and Vice-President of the Chicago and Northwestern Railway Company, and President of the Union Pacific Railroad Company, as a Consulting Engineer on the Chicago Smoke Abatement and Electrification Commission. He then became Consulting Engineer for the Southern and Frisco Railway Systems on the construction of the Chalmette Slip, a deep waterway and terminal at New Orleans, La. On the completion of this work in 1915 he retired from active practice, and devoted the remainder of his life to his personal interests in Chicago, and his country home on the Delaware River, in Pennsylvania.

Mr. Evans was a member of the Western Society of Engineers and of the Zeta Psi Fraternity.

He was married on June 2, 1879, to Frances Helen Robinson, at Mast Hope, Pa., and is survived by his widow and two children, Mrs. E. T. Hiscox, of Tyringham, Mass., and Major E. Webster Evans, of Detroit, Mich.

Mr. and Mrs. Evans celebrated their Golden Wedding Anniversary on Sunday, June 2, 1929, at Mast Hope. Mr. Evans was apparently in fine health at that time, but a sudden heart attack in his sleep, early on Wednesday morning, June 5, caused his death.

His sterling character, his love of justice to all, his enthusiasm in his work, and his encouragement to others under trying circumstances, endeared him to the many who worked with him, or came in contact with him.

Mr. Evans was elected a Member of the American Society of Civil Engineers on July 3, 1889.

## ROBERT JOHN HARDING, M. Am. Soc. C. E.*

DIED JULY 31, 1929.

Robert John Harding, the son of Albert G. and Mary A. (Peck) Harding, was born on February 16, 1880, at Allegan, Mich. Soon afterward, however, his people moved to Chatham, N. Y., and this town was his early home.

Mr. Harding was graduated from Cornell University at Ithaca, N. Y., in the Class of 1903. After serving as an Instructor at the University of Pennsylvania, and as Recorder on the United States Lake Survey, he became Superintendent of Public Works in Hudson, N. Y., in 1905. In 1908, he accepted a similar position in the larger city of Poughkeepsie, N. Y., where he rendered distinguished service until 1912. During the latter year he was engaged in private practice in Poughkeepsie.

In 1913, he went to San Antonio, Tex., as Manager of the Water Supply System which was owned by Belgian interests. He managed the extensive property for the foreign owners with ability and fidelity during the troublesome years of the World War.

After the war, the Belgian stockholders desired to realize on their investment and Mr. Harding formed a group among neighbors to purchase the stock and take over the property. As Chief Engineer and Vice-President, he continued to manage and extend the property, known as the San Antonio Water Supply Company, until it was sold to the City of San Antonio in 1925.

After the sale of the water-works, Mr. Harding was engaged principally in the development of several small water companies, but he made a professional trip to South America and, later, assisted in the investigation of sewers in the Borough of Queens in the City of New York.

Mr. Harding was interested in Red Cross work and was Chairman of the San Antonio Committee during the war period. He had a keen insight and was rarely wrong in his judgment of the men and conditions with which he had to contend. He was quiet and unassuming, but forceful withal—a competent executive and a loyal friend.

He was fond of hunting and fishing and was a most delightful companion on camping expeditions in search of wild ducks and turkeys. One of the writer's most cherished memories is of a long automobile trip from Texas to Arizona, with visits to all the most important dams along the route.

Mr. Harding was a member of the American Water Works Association, the New England Water Works Association, and the American Public Health Association. In the prime of his active manhood, and with much to look forward to, he was killed in an automobile accident for which he was in no way responsible.

He was married on December 25, 1903, to Louise E. Swick, of Merklenburgh, N. Y., who survives him, with three sons, two daughters, and one grandchild.

Mr. Harding was elected an Associate Member of the American Society of Civil Engineers on November 4, 1908, and a Member on September 3, 1912.

* Memoir prepared by Allen Hazen, M. Am. Soc. C. E.

## WILLIAM DEAN JANNEY, M. Am. Soc. C. E.*

DIED OCTOBER 30, 1925.

William Dean Janney, the son of William Warder and Marion (Dean) Janney, was born in Baltimore, Md. on September 8, 1859, of English and Scotch-Irish ancestry. After his preliminary education had been completed at the Friends School in Baltimore, he attended Lehigh University at Bethlehem, Pa., for two years.

In July, 1881, Mr. Janney entered the employ of the Baltimore and Ohio Railroad Company, and was engaged on preliminary surveys and location. In November of that year, he was appointed Assistant Engineer with the Norfolk and Western Railway Company, later being placed in charge of the masonry construction on 14 miles of line, which included 2 miles of road extending east from Pocahontas, Va. This work involved masonry for 17 bridges, 100 or more culverts, trestle footings, grading trestles, two tunnels, and continuous track-laying.

In April, 1883, he became General Agent for the Lexington Manufacturing Company, but he soon resigned this position to re-enter engineering. During the winter, he attended classes in Freehand and Architectural Drawing at the Maryland Institute of Art and Design, and, in the spring, he entered the employ of the Annapolis and Baltimore Short Line Railroad Company, which he served as Instrumentman and, later, as Chief Engineer. In this capacity, he completed surveys, made up estimates of cost, designed structures, financed the enterprise, and built the major part of the line.

Three years later, however, Mr. Janney resigned this position to return to the Norfolk and Western Railroad Company. He remained in the service of this road and its affiliated interests until 1894. He was placed in charge of both preliminary and location surveys, which included several hundred miles of construction work, with division terminals, yards, and shops, and other railroad construction of a heavy type. One difficult problem of which he had charge was the foundation work for a bridge across the Ohio and Kenova Rivers in West Virginia. During this period, he was also engaged in the construction of office buildings, the planning of town sites, the erection of dwellings, the grading of streets, the design and construction of a sewerage system, etc. This series of undertakings involved the design and construction of work amounting to several millions of dollars.

In 1891, he was appointed President and General Manager of the Wells Branch Coal Company, for the purpose of conducting the initial development of a new field. This position was created for him by the President of the Norfolk and Western Railroad Company, with which road he had so long been associated. On the recommendation of the Chief Engineer he selected the site, consummated the lease, constructed the necessary works, developed the mine, and at length put it into active operation. When he considered that it had fulfilled its purpose he sold it to private individuals.

* Memoir prepared by A. E. Christhilf, M. Am. Soc. C. E.

From 1894 to 1897, Mr. Janney was engaged in private practice at Ceredo, W. Va., where he undertook and completed contracts for several railroads, foundations for buildings, surveys and general plans for a bridge over the Ohio River at Ashland, Ky. He likewise examined and made reports on timber and coal lands, conducting expert investigation of territory of obscure location and defective description which he often succeeded in retrieving for the owners. When the celebrated "King Land Case" appeared in Court, he acted as leading engineering expert by appointment of counsel for the defense, and aided in the preparation of cases, testifying at the various trials. He finally prepared all the maps used in the presentation of the case before the Supreme Court of the United States in which the decisive victory was gained.

He was employed during 1896 and 1897, with the United States Engineer Department as Assistant Engineer on surveys of the Ohio River from Pittsburgh, Pa., to Marietta, Ohio, to determine locks and dams for slack-water navigation; later, he was in charge of the Drafting Department and became Acting Geologist for party. In the fall of 1897, he was engaged as Chief Engineer with the W. M. Ritter Lumber Company of Welch, W. Va., and Columbus, Ohio, in charge of all surveys, laying off of timber cuttings, and the location and construction of saw-mills. He developed railroads and other means of transportation for logging operations, and determined the measurement of obscure and disputed boundary lines.

From December, 1900, to April, 1904, Mr. Janney acted as Chief Engineer of the Northern Railway Company, of Costa Rica, for the United Fruit Company. At this time, he was in charge of the location and construction of railroads, shops (including the selection, purchase, and installation of all machine tools and appliances), station buildings, bridges, and large apartment and office buildings of steel concrete.

In July, 1904, he became Chief Engineer for the Elk River Coal and Lumber Company, and for the Buffalo Creek and Gauley Railroad Company in connection with the development of about 80 000 acres of coal and timber land in Clay and Nicholas Counties, West Virginia. After he had built the railroad into the property, designed all the structures, and reported on timber holdings and coal deposits, he started the actual development of the mines. Later, he returned to Baltimore to accept a position as General Superintendent of Construction in the building of new reinforced concrete piers for the City.

In 1911, when this work was nearing completion, Mr. Janney was engaged by D. B. Banks, Consulting Engineer, of Baltimore, to take charge of work in connection with a high-pressure system of water supply in West Virginia. In this capacity he made a topographic survey of a stream valley to determine impounds and location of dams, and, later, he located 43 miles of pipe line. He conducted all negotiations for the acquisition of lands, etc.

From 1912 to 1919, he was engaged in private practice with Mr. Banks, making examinations and reporting on various hydro-electric projects, waterworks, and irrigation systems. This work consisted also of reconstruction, involving intricate engineering and construction problems, hydrographic surveys, layout of channel dredging, and the preparation and testifying in many cases of litigation on damage suits.

During the first year of the World War Mr. Janney served as Assistant Engineer with the Quartermaster Corps, U. S. Army, in charge of the distribution and allotment of railroad materials, but soon after, he was forced to resign on account of ill health.

In December, 1918, he was again engaged in private practice, and subsequently until his death acted as Special Engineer with the United Railways and Electric Company of Baltimore.

Mr. Janney was a man of high character and was well liked by all his associates. He was a member of the Engineers Club of Baltimore, and for several years the Editor of the monthly *Journal* for that Society. He was intensely interested in the organization of The Boy Scouts of America, and was a member of the Court of Honor in awarding badges. He was also an Elder and Trustee of the Second Presbyterian Church of Baltimore.

He had few recreations, but took great delight in tinkering around his own home, or in fishing, spending many hours with his rod in hand waiting for the quick jerk which told of an unsuspecting nibble.

He was married in Baltimore on February 16, 1886, to Marion Rowe; they had five children, Marion, William Rowe, Gertrude, John W., and Charles Homer.

Mr. Janney was elected an Associate Member of the American Society of Civil Engineers in 1891, and a Member on April 1, 1896.

## CHARLES LEWIS POTTER, M. Am. Soc. C. E.*

DIED AUGUST 6, 1928.

Charles Lewis Potter, the son of Benjamin R. and Susan E. (Smullen) Potter, was born at Lisbon Falls, Me., on January 24, 1864. His boyhood was passed on a farm in his native State and his early schooling was obtained in the public schools in the vicinity.

Mr. Potter entered the United States Military Academy at West Point, N. Y., in 1882, and was graduated therefrom in 1886, standing fifth in his Class. He was commissioned a Second Lieutenant and assigned to the Fifth Cavalry, on July 1, 1886. He served on frontier duty at Fort Supply, Indian Territory, in September and October, 1886, and then in garrison at Fort Leavenworth, Kansas, until March 29, 1887. He was transferred to the Corps of Engineers, on February 2, 1887, and was sent to the Engineer School of Application at Willets Point, N. Y., from which he was graduated in 1889.

Lieut. Potter's first Engineer assignment came in October, 1889, when he was appointed Assistant to the Engineer Officer in charge, at Montgomery, Ala., on the improvement of rivers in Alabama, Georgia, and Florida. His subsequent assignments were distributed over a great part of the United States as follows: Fortifications in San Francisco Harbor, and rivers in Southern California, 1891-1897; rivers in Oregon, 1897-1898; the Mississippi River at Memphis, Tenn., 1900-1903; the Great Lakes, Duluth, Minn., 1903-1906; Porto Rico, as Chief Engineer, Lighthouse Service, 1907-1910; the Mississippi River,

* Memoir prepared by William S. Mitchell and James W. Skelly, Members, Am. Soc. C. E.

at St. Louis, Mo., 1910-1912, and at St. Paul, Minn., 1912-1915; rivers and harbors at Portland, Ore., 1915-1916; at Boston, Mass., 1916-1917; at San Francisco, Calif., including fortifications, 1918-1920; and, finally, as President of the Mississippi River Commission, at St. Louis, from March 19, 1920, to June 12, 1928.

He was promoted to the rank of Captain on July 5, 1898; Major, September 14, 1904; Lieutenant-Colonel on February 27, 1911; and Colonel on November 27, 1916.

During the Spanish-American War and the Philippine Insurrection, Colonel Potter was Chief Engineer, in the Philippines, of the 8th Army Corps, on the Staffs of Generals Merritt and Otis (1898-1899), and during the World War he was Director of Gas Service (now the Chemical Warfare Service) at Washington, D. C. (1917-1918).

Colonel Potter's outstanding work was as President of the Mississippi River Commission. He was a man of very attractive personality, cordial and frank. He readily secured the confidence and respect of the leading men with whom the Commission had to deal, which had a far-reaching effect on harmonizing its many interests. During this service he was also Division Engineer of the Western Engineer Division, comprising river improvement works in the Engineer Districts in the Mississippi Valley from the head-waters to the Red River, including the rivers tributary to the territory of the Commission, except the Ohio and Illinois Rivers, and the Mississippi below Red River.

His greatest work came in 1927, following the phenomenal and disastrous flood of that spring in the Lower Mississippi Valley. During the months following this flood, to the end of 1927, he labored unceasingly in the preparation of comprehensive plans for the control of this great river, to be submitted for the consideration of Congress, frequently appearing before its Committees in advocacy of the Commission plan for the solution of the great problem. There is no doubt that the strain under which he labored so long and so assiduously, weakened and undermined his health.

On reaching the military age for retirement, on January 24, 1928, he was retired, but was immediately recalled to active service, continuing as President of the Mississippi River Commission. The Flood Control Act of Congress, pending at the time, carried the provision that the President of the Mississippi River Commission should have the rank of Brigadier-General. Recognizing the importance of the Commission and of Colonel Potter's great service thereon, he was promoted to that rank on May 15, 1928.

Deeply interested in the vast work to be carried on by the Commission, he looked forward with eager anticipation to its prosecution. His wish was not fulfilled. To his great surprise and disappointment he was relieved from active duty on June 12, 1928, and shortly after was forced to undergo what was expected to be a comparatively safe surgical operation. His impaired physical condition, however, left him unable to rally from the shock of the operation, and he passed away quietly, on August 6, 1928.

General Potter made readily, and retained loyally, friends in every walk of life. He loved Nature and the outdoors. He was a keen student of archæological and historical matters and of fraternal subjects concerning

which he possessed a fine library. He contributed numerous articles to engineering journals concerning the river problems before him. These were widely read and appreciated for sound engineering wisdom, great common sense, and clarity of statement.

He was a member of many Masonic bodies, having joined the organization in Memphis, where he was advanced to the Thirty-third and last Degree of the Ancient and Accepted Scottish Rite in a most unusually short time. In spite of frequent changes of station incident to Army life, General Potter was one of the few officers ever to attain this distinction. His funeral services were held in the Scottish Rite Cathedral at St. Louis, and his ashes were deposited in Valhalla Mausoleum near-by.

General Potter is survived by his widow, Mrs. Sophie H. Potter, and his step-daughter, Caralisa Nichols.

General Potter was elected a Member of the American Society of Civil Engineers on April 1, 1903.

## GEORGE FETTER STICKNEY, M. Am. Soc. C. E.*

DIED JANUARY 27, 1929.

George Fetter Stickney was born on January 24, 1869, in St. Paul, Minn., the son of Amos Stickney and Virginia (Fetter) Stickney. His boyhood days were spent in various cities where his father, who was a Major and later a Brigadier General in the Corps of Engineers, United States Army, was stationed in charge of river and harbor improvements. He received the degree of Bachelor of Philosophy from the Sheffield Scientific School of Yale University in 1891, where he was a member of Chi Phi Fraternity.

Mr. Stickney's first employment was with the Mexican International Railroad Company. From 1891 to 1893, he served as Rodman, Leveler, and Resident Engineer retracing location surveys on the Durango Extension, supervising the construction of 30 km. of line, including yards and freight and passenger depots in the City of Durango, Durango, Mexico, making reconnaissance and preliminary surveys to Rio Tunal and supervising the grading of 25 km. of the Sierra Mojada Extension.

He entered the Government Service in 1893 as United States Assistant Engineer in charge of repairs on locks and dams on the Kentucky River in Kentucky, and made surveys for the location of Lock No. 7 at High Bridge, Ky., building the lock-gates and designing boats, barges, dwellings, etc. From 1895 to 1898, he was in local charge of the operation and care of five locks and dams, a dredge and a snag-boat on Green and Barren Rivers and a lock on Rough River, Ky. In addition, he rebuilt part of Dam No. 2 and finished the construction of Lock No. 2 at Rumsey, Ky.

Mr. Stickney served for nearly two years in the United States and Cuba during the Spanish-American War as Captain in command of Company C, Third Regiment, United States Volunteer Engineers. In 1899 and 1900 he

* Memoir prepared by Joseph Ripley, M. Am. Soc. C. E.

made a survey for the location of a terminal railroad at Sabine Pass, Tex.; graded streets on the outskirts of Paducah, Ky.; and made plans for the improvement of Jefferson Barracks, Missouri, with layouts for sewers, retaining walls, roads, and the grading of the parade grounds.

In 1901 he made the surveys and designed the masonry for a bridge for the Mississippi River and Bonne Terre Railroad Company in Southeastern Missouri. In 1901 and 1902 he had charge as Assistant Engineer, for the Lake Superior Power Company, of constructing the "compensating works", a movable dam of five large Stoney gates, at the head of the rapids in St. Mary's River, at Sault Ste. Marie, Ont., Canada. He also superintended the construction of piers and abutments of the Louisville and Nashville Railroad Bridge across the Alabama River at Selma, Ala., for Paterson, Collier and Company, Contractors.

In 1903, Mr. Stickney superintended the construction for the J. S. Paterson Construction Company of a concrete arch viaduct forming the east approach to the Southern Illinois and Missouri Bridge, at Thebes, Ill. In 1904 and 1905, as U. S. Assistant Engineer, he was employed on harbor work in Northern New Jersey and along Long Island Sound. For nine years, from 1905 to 1914, Mr. Stickney was employed in the Department of the State Engineer and Surveyor of New York, as Supervising Engineer, in connection with the Barge Canal; and in addition he had general charge of the design of large sections of the New York State canals, namely, the Erie, Champlain, Oswego, and the Cayuga-Seneca Canals, together with the supervision of construction on more than 250 miles of waterway, including approximately one-third of all the canal structures, the construction cost of which exceeded $35 000 000.

Another important engagement was that of Consulting Engineer to the Lake Erie and Ohio River Canal Board of Pennsylvania from 1915 to 1917. As such Mr. Stickney had charge of preliminary investigations, surveys, preparation of plans, estimates, etc., for the proposed waterway which was estimated to cost $100 000 000. This latter work included the canalization of the Beaver and Mahoning Rivers, in Pennsylvania and Ohio, and the construction of a canal from Niles, Ohio, to a point on Lake Erie just west of Ashtabula, Ohio, with locks, dams, and reservoirs, as well as port developments at Lake Erie and at various cities along the route of the canal.

As a Consulting Engineer Mr. Stickney maintained an office in Albany, N. Y., from 1917 to 1928, and made many hydraulic investigations for water-power developments and river improvements. He also was an expert witness in several claim cases involving water rights. He was the inventor of the siphon spillway for discharging water through dams, and of an automatic movable crest for dams. He contributed a paper entitled "Siphon Spillways"* to the *Transactions* of the Society, as well as articles on engineering subjects to various technical publications.

Mr. Stickney was a man of keen intellect and had the faculty of quickly sifting out the essential elements of a problem and the ability to work out in detail an accurate solution. His skill in sketching rapidly and accurately with

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1098.

a few lines materially aided in making plain his conception of the subject or of an object. He was very methodical and kept his records in precise and complete form. He was self-reliant and on occasion was tenacious in holding to his views. Always he was a thorough gentleman, with an ever-present quiet dignity. He had been in declining health for three or four years before his death, the cause of which he had attributed to heart trouble, not knowing he had cancer of the stomach. He was buried at West Point, N. Y.

In 1894 he was married to Katherine Halloran, of Frankfort, Ky., and is survived by his widow, two children, Josephine Stickney and George Fetter Stickney, Jr., and by his mother and two sisters, all residing in New York, N. Y.

Mr. Stickney was elected a Junior of the American Society of Civil Engineers on May 1, 1894; an Associate Member on April 5, 1899; and a Member on September 4, 1906.

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## CHARLES CLIFTON UPHAM, M. Am. Soc. C. E.*

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DIED JANUARY 7, 1928.

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Charles Clifton Upham was born on May 5, 1852, of good old New England stock. He was of the seventh generation, in direct line of descent, from John Upham who came from England in 1635 with a colony of twenty-one families under the leadership of one, Joseph Hull. They settled in what was then called Wessaguscus, afterward named Weymouth, in Massachusetts. John Upham was evidently a man of consequence in his community having been a Delegate to the General Court as well as Selectman of the Town. He was one of six who treated with the Indians for the lands of Weymouth, the settlers previously having occupied this site as squatters. Later, in Malden, Mass., to which he soon moved, John Upham served as Selectman as well as Moderator of the Town Meeting. His son, Lieut. Phineas Upham, who gained his title in the Colonial Wars, was also a man of prominence. It was said of him that "Worcester [Massachusetts] owes its foundation in no small degree, as it appears, to his activity and energy."

Other Uphams in the direct line of descent appear to have been men of prominence and enterprise. Several of Mr. Upham's ancestors, not of that name, however, came to Massachusetts in the early half of the Seventeenth Century. His father's mother was a *Mayflower* descendant. His father, born in Castine, Me., had moved to Boston, Mass., and was one of a party of ten to sail from there for California, around Cape Horn, in 1849. After eighteen months, he returned and lived in Woburn, Mass., where Charles Clifton Upham was born. In 1853, the family moved to Dixon, Ill., where young Upham was brought up. The father was recognized in Dixon as a man of education and of excellent literary taste. Charles Clifton Upham had a fine family heritage.

Beginning at eighteen years of age, from 1870 to 1872, Mr. Upham was Assistant Engineer on the Illinois River Improvement, Slackwater Navigation. The next three years found him gaining his first experience in railroading as

* Memoir prepared by C. Frank Allen, M. Am. Soc. C. E.

Assistant Engineer of the Grand Junction Railroad and the Belleville and North Hastings Railroad, in Ontario, Canada. Then, for three years he was in Del Norte, Colo. During at least part of this time, he was expecting to install lixiviation works for reducing ores, but this project never materialized.

Meanwhile, he qualified as a United States Deputy Mineral Surveyor. He enjoyed relating that, while he was in Del Norte awaiting developments, he and a friend, who later became Cashier of the local bank, worked with pick and shovel on highway construction, and that he loaned the contractor money when he needed it to complete his contract.

For several years Mr. Upham was with the Atchison, Topeka, and Santa Fé Railroad Company on location and construction, having served at one time as Transitman on part of the difficult location of the line through the Grand Canyon of the Arkansas. When the Mexican Central Railroad was started from Paso del Norte (now Juarez) south toward Chihuahua, he became Locating Engineer, there being then no Chief Engineer. After several hundred miles had been located, he was offered the berth of Chief Engineer at the same salary, $250 per month. This he refused as he deemed the salary inadequate. He was much needed on the work and did not hesitate to remain as Locating Engineer, the salary being then, as he thought, adequate for that position. This was an example of the refusal of desirable advancement because of an honorable respect for the dignity of his profession—and this on the part of a man not above working with pick and shovel, at a time when he had ample funds in his pocket. Mr. Upham needed no formal code of ethics; instinctively, he steered straight.

By that time he had acquired a comfortably ready use of the Spanish language and was very popular with the various Mexican officials at Paso del Norte. He was conspicuously an asset to the railroad. It was at this time that his locating party was wiped out by a band of Yaqui Indians on the war-path, and it was first thought that he was with the party; but as it developed later he happened to be en route elsewhere. His obituary had been published, however. When the news reached Paso del Norte, his Mexican friends were most seriously disturbed and it was surprising how quickly the cavalry regiment was put in motion, the Colonel at its head. With Mr. Upham in danger, it was a matter of personal friendship rather than a routine sortie.

Following this, Mr. Upham very soon was offered and accepted the post of Resident and Chief Engineer of the Tampico Branch of the Mexican Central Railway, extending from Tampico to San Luis Potosi. This railroad he located and constructed. It was in very heavy work and very difficult location for which, no doubt, his experience in the Grand Canyon of the Arkansas especially qualified him. He was able to improve very materially on the location previously made by another engineer.

When this work was completed, he became, in 1884, Chief Engineer of the Colorado Railway Company, with headquarters at Denver, Colo. Shortly afterward he was appointed Chief Engineer of the Chicago, Burlington, and Northern Railroad Company, then under construction, with headquarters at St. Paul, Minn. Following this, he was appointed General Superintendent of all the lines in Illinois, of the Chicago, Burlington, and Quincy System.

Temporary ill health resulting from exposure in the great strike on that System caused him to relinquish this post.

His health recovered, Mr. Upham was, in 1895, a Consulting Engineer, as well as Chief Engineer and General Manager of the Salt Lake Rapid Transit Company, operating a 30-mile electric line. Later, he was General Manager of a 60-mile electric street railway at Lincoln, Nebr., in which he also had some financial interest.

Somewhat later, Mr. Upham moved to New York, N. Y., where he became Consulting Engineer for the New York Steam Heating Company, and subsequently served as its Vice-President and General Manager; he remained in that position until ill health caused him to retire several years before his death which occurred in New York on January 7, 1928. While he was engaged with this Company he made an extensive study of methods of pulverizing coal and transporting it by water carriage through pipes from the mines to large cities. Nothing tangible has resulted, however, whether because he was ahead of the times, or because of inherent difficulties in the process, it is impossible to say.

Mr. Upham was a member of the Sons of the American Revolution; the Society of Colonial Wars; and the Society of Mayflower Descendants.

He was a man of fine presence, tall, blonde, well built, physically and socially attractive, and of fine character; a gentleman in appearance, manner, and substance; he made many friends. As reflecting in some degree the influences which formed his character, his brother who compiled the Upham Genealogy which has supplied some of the foregoing data, in closing wrote:

"If its effect is * * * as I have hoped it may be * * * to cause each one bearing the name of Upham to feel that he has responsibility for bearing it creditably, then indeed will a grand object have been attained, and we may adopt the sentiment of the New Brunswick Uphams:

"'If it is not in all mortals to command success, we will do more, deserve it.'"

Charles Clifton Upham qualified; he deserved success; he also secured it. The writer of this memoir, who was associated with him in Mexico and in New Mexico, holds him in fond remembrance.

He is survived by his wife, Anna St. John Eells, to whom he was married in December, 1883, and who also traced her ancestry to the early colonists of New England.

Mr. Upham was elected a Member of the American Society of Civil Engineers on April 7, 1897.

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## ROBERT HARLOW ANDERSON, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 8, 1929.

Robert Harlow Anderson was born in North Woburn, Mass., on October 2, 1877. He was educated at Robert College, Constantinople, Turkey, at

* Memoir prepared by J. A. Longley, Esq., Chattanooga, Tenn.

Williams College, Williamstown, Mass., and at Cornell University, Ithaca, N. Y., receiving the degree of Civil Engineer from the latter institution in 1898.

In June, 1898, Mr. Anderson began his professional career as Rodman and Draftsman on the United States Deep Waterway Survey through New York State. From December to March, 1900, he served as Draftsman for James H. Fuertes, M. Am. Soc. C. E., on an investigation of the city water supply for the Merchants' Association of New York City. In March, 1900, he was engaged as Draftsman and Transitman on surveys for the Nicaragua Canal and, in 1901, became Transitman for the late George S. Morison, Past-President, Am. Soc. C. E., on water power investigations on the Susquehanna River. From 1903 to 1904 he was with the United States Geological Survey and in conjunction with John C. Hoyt, M. Am. Soc. C. E., prepared a report on the hydrography of this river.* After the completion of this report he was appointed Assistant Engineer with the Isthmian Canal Commission. Mr. Anderson was one of the first engineers on the Panama Canal. He was engaged in making investigations and plans, under the Resident Engineer at Bas Obispo, Panama, for the control of the Chagres River.

From 1905 to 1909 he served as Engineer for the McCall's Ferry Power Company on the Holtwood, Pa., Power Dam, and he was also employed on work in connection with an irrigation project at Twin Falls, Idaho, and for a hydro-electric project in Nova Scotia. Mr. Anderson's first connection with the Tennessee Electric Power Company was in 1909 when, as Resident Engineer for the J. G. White Construction Company, he was engaged on the construction of two hydro-electric developments for the Power Company at Parksville and Caney Creek, on the Ocoee River. From 1915 to 1917 he was located at Rock Island, Tenn., in charge of the construction of the Great Falls Hydro-Electric Development for the same Company, and, from 1919 to 1921, he was Engineer for the J. G. White Construction Company, located at Rifton, N. Y., during the building of the dam and power plant on the Wallkill River. He also was engaged on other engineering work for the J. G. White Construction Company at Langley Field, Virginia, and at the U. S. Nitrate Plant at Muscle Shoals, Alabama.

In 1922, Mr. Anderson returned to The Tennessee Electric Power Company as Hydraulic Engineer, which position he held at the time of his death. During 1923 and 1924 he was in charge of the construction of additions to the dam and power-house at the Great Falls Plant. Subsequently, he made surveys and engineering investigations of various projects and was in charge of the construction of an earth storage dam on the Toccoa River, for the Toccoa Electric Power Company, a subsidiary of The Tennessee Electric Power Company. This dam is to be 167 ft. high and will contain approximately 1 600 000 cu. ft. of earth, with a power installation of 25 000 h.p. Mr. Anderson was actively in charge of this work until his death.

Mr. Anderson is survived by his widow, Mrs. Elizabeth Fry Anderson, of Sunnyburn, Pa., to whom he was married in 1907. He also leaves one brother, Roger H. Anderson, an Attorney of New York City, and two sisters, Mrs. George Baker, of Altadena, Calif., and Sarah Anderson, of Pasadena, Calif.

* *Bulletin No. 109*, U. S. Geological Survey.

His father, Charles Anderson, who died in 1928, was Dean of Robert College, Constantinople, until his retirement in 1917, when he returned to the United States.

Mr. Anderson leaves a host of personal friends, both within and without The Tennessee Electric Power Company, who held him in deep admiration for his sterling qualities as a man and a most capable engineer.

Mr. Anderson was elected an Associate Member of the American Society of Civil Engineers on January 8, 1908.

## THOMAS McKENZIE, Assoc. M. Am. Soc. C. E.*

DIED JULY 25, 1929.

Thomas McKenzie, the son of Andrew and Martha (Cook) McKenzie, was born in Providence, R. I., on September 10, 1867. His parents were of Scottish and English ancestry, his father's family having come to the United States in their youth. His mother died in his childhood and he went to live with his uncle and aunt, Mr. and Mrs. John McCulloch, of Providence, R. I.

While Mr. McKenzie was still a young man his uncle and aunt removed to Westerly, R. I., where he attended the local public schools and, later, learned the moulder's trade in the shops of the C. B. Cottrell and Sons Press Works, in Pawcatuck, Conn., situated across the river from Westerly. His uncle, who also was employed there as a Master Moulder, later, decided to enter the business field in Westerly for himself, taking his young nephew with him. Mr. McKenzie continued in business with his uncle until 1889, when he determined to abandon foundry work and enter the engineering and building profession by means of which he might work his way upward in life to a more satisfactory and lucrative career.

In January, 1889, he went to Providence, and entered the office of the late J. Herbert Shedd, M. Am. Soc. C. E., as a student in civil engineering. He remained with Mr. Shedd for three years, acting as Rodman, Chainman, and Office Draftsman. During his studentship he attended the night schools in Providence, employing his spare time improving his natural talents as a Draftsman, in which work he later became very proficient.

While Mr. McKenzie was with Mr. Shedd, from 1889 to 1892, he was engaged on the encroachment line surveys on the Pawcatuck River at Westerly, and Stonington, Conn., as well as on other engineering work in connection with Mr. Shedd's general practice.

After completing his three years' apprenticeship, he found it necessary to make a change of employment, owing to Mr. Shedd's connection with the City Engineer's Office in Providence. Consequently, early in 1892, he entered the office of the late Samuel M. Gray, M. Am. Soc. C. E., as a Construction Engineer. Mr. McKenzie remained with Mr. Gray for five years as an Assistant Engineer. During this time he was in charge of construction at Bradford, Vt., Chautauqua, N. Y., Meredith, N. H., Gallipolis, Ohio, and at Rockville, Conn.

* Memoir prepared by George H. Leland, M. Am. Soc. C. E.

His work at Bradford consisted of a small water-works system, including a storage and distributing reservoir. At Chautauqua, he completed the sewer system already well under way at the time of his arrival on the ground. This work included the building of a Chemical Precipitation Disposal Plant, which kept him busily engaged all of one winter. This was followed by the building of a water-works system, including a small storage and distributing reservoir, with its earth work dam and masonry spillway, at Meredith. At Gallipolis, he constructed the water-works plant, including a system of pipes and hydrants, reservoir, pumping station, and a unique source of supply. This consisted of sinking four 4-ft., cast-iron pipe wells on an island in the Ohio River. These wells were connected with the pumps by a long line of suction pipe located on the high banks of the river. Considerable difficulty was experienced in the construction of these works, but they were finally completed in a satisfactory manner. His work at Rockville was the building of a sewer system in 1895 and 1896. This construction required considerable rock excavation in the streets and an inverted siphon under the Hockanum River.

In the construction of these various undertakings, Mr. McKenzie was very careful and conscientious to see that all parts of the work were done in accordance with the contracts and specifications.

In 1897, the Town of Westerly, after considerable litigation in the Courts, acquired the rights, franchises, good-will, and title to all the property held by the Westerly Water Company, including the right to use the waters of Shunoc Brook in Connecticut as a source of water supply and the privilege of furnishing water to the Village of Pawcatuck, Conn.

Under the direction of Mr. Gray, Consulting Engineer, plans were made to install a system of small wells driven in the Pawcatuck Valley. These plans involved the abandonment of Shunoc Brook as a water supply, and necessitated a new pumping station and new mains. Mr. McKenzie was selected to take charge of the reconstruction of the new and improved system. After completing the improvements and additions, the system was given a thorough test, found to be satisfactory, and the works were then turned over to the Town. The Board of Water Commissioners immediately appointed Mr. McKenzie to be Superintendent and General Manager for the Westerly Water-Works which position, extending over a period of more than thirty-two years, he held at the time of his death.

During his early connection with the water-works, or in 1902, the pipe system was extended to the Watch Hill District and along the South Shore, supplying the summer population with good water and fire protection. The extension of the distribution with the ever-increasing consumption of water on both sides of the river required more storage capacity than was available from the one small iron stand-pipe, built in 1888 by the Water Company. As a result, one of the most important improvements in the system was the building of a reinforced concrete stand-pipe under the supervision of Mr. McKenzie, with Mr. Gray as Consulting Engineer. The plan was successfully completed by the Aberthaw Construction Company, of Boston, Mass., and was put into service in connection with the iron stand-pipe then in use.

In 1914 the Town decided to install a sewer system for the Westerly Fire District and Mr. McKenzie was instructed to take charge of the surveys for this work in connection with his other duties as Superintendent of the Water-Works. The sewer system as finally planned for the District included in addition to the pipe system, two pumping stations, an Imhoff tank, and two filter beds, costing approximately $600 000. These works were built largely by contract under the supervision of Mr. McKenzie as Engineer in Charge.

In connection with his duties attending the water-works and sewer system of Westerly, he found time to rebuild some of the streets and highways for the town, also, to assist in an advisory capacity some of the near-by towns and villages with their water problems. He was an indefatigable worker, and was always willing to serve in the interests of the town's affairs, and, later, in the building of the new Memorial Hospital in Westerly, for which he served on the Building Committee.

Mr. McKenzie was prominent in Masonic circles, having reached the Thirty-second Degree several years previous to his death. He was Grand Master of the Connecticut Grand Lodge. In 1916, he was Connecticut State Chairman of the George Washington National Memorial Association, besides holding many other important offices in Rhode Island and Connecticut. He was a member of the New England Water Works Association and the Providence Engineering Society. He was a communicant of Christ Protestant Episcopal Church, of Westerly, a member of the Vestry, Junior Warden, and Treasurer of the Church organization.

In 1892, while working on his first construction job with Mr. Gray at Bradford, Vt., he met Gertrude F. Jones, daughter of the late Dr. Julian F. Jones. They were married in Bradford, Vt., in January, 1896, and made their home in Providence until Mr. McKenzie went to Westerly in 1897. His wife died at her home in Westerly on September 28, 1928, leaving one son, Julian L., 22 years of age.

Mr. McKenzie felt the loss of his companion very keenly, but bore up bravely and plunged into his work far beyond his strength, which over-activity brought on a severe attack of heart trouble terminating in his death after four days' illness. He always had been of robust health and unusually well and strong, but he over-taxed himself.

He was very systematic in all his work; careful statistics and records were kept of the operation of both the water and sewer plants, carefully indexed, and put on file, together with the plans of all the works. He always managed to surround himself with a corps of faithful assistants and co-laborers, who at all times were willing and ready to perform the tasks he gave them. Many of these assistants and laborers remained with him for several years. Mr. McKenzie was greatly respected and admired as a faithful and loyal servant of the community in which he served, was popular as a speaker at public meetings, and an able executive and presiding officer. His loss will be greatly felt in the town in which he spent the best part of his life.

In closing, it may be well to remark that Mr. McKenzie owed a large measure of his success and formation of character to his Aunt Mary who cared for him in his early life in place of his mother and of whom he was very

fond. His own mother could not have done more for him. Later, he repaid his debt of gratitude and love to her by taking her into his own home in her declining years.

Mr. McKenzie was elected an Associate Member of the American Society of Civil Engineers on November 6, 1895.

## THOMAS PATTON STEVENSON, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 8, 1928.

Thomas Patton Stevenson was born in Philadelphia, Pa., on May 6, 1876. In 1897, he matriculated at the University of Pennsylvania where he studied Civil Engineering for two years.

In May, 1900, Mr. Stevenson went to Cuba as Assistant with the Engineer Corps of the Cuban Steel Ore Company on the construction of ten miles of railroad; from August, 1900, to February, 1902, he was engaged with the Engineering Department of Santiago, Cuba, (under Capt. S. D. Rockenbach, U. S. Corps of Engineers, Engineer Officer, and the late A. S. Hobby, Jr., M. Am. Soc. C. E., Principal Assistant Engineer) in surveying, road and culvert construction, and sewer work.

In 1902, he organized the firm of Stevenson and Headman, Engineers and Contractors, of Philadelphia, Pa., and Rio de Janeiro, Brazil. In March, 1902, the firm laid by contract 4 000 m. of pipe and connection with stand-pipe in the City of Paramaribo, Dutch Guiana; in 1905, it built by contract 6 km. of railroad embankments, 100 km. in the interior of Dutch Guiana on a system known as "Suriname Koloniale Spoorwegen", and also signed a contract with the "Koloniale Spoorwegen" for railroad ties (native wood).

Having liquidated his firm, Mr. Stevenson went to Para, Brazil, in 1906, as the Construction Representative of an American Syndicate holding concessions there. In this same year, he went to Rio de Janeiro where he held a position as Assistant Engineer in the Rio de Janeiro Tramway, Light, and Power Company, which had just been organized with the object of supplying electric tramway power and light service to that city. He remained in this position until 1912 when he founded the Federal Express Company in Rio de Janeiro, of which Company he served as President until his death.

During the last sixteen years of his life, Mr. Stevenson demonstrated an extraordinary capacity for work and organization. He led the Federal Express Company which he formed with American and Brazilian elements and capital to sound prosperity, thus giving an example worth imitating, with respect to efficient co-operation of American interests in the development of Brazil.

The Federal Express Company, which continues its successful operation under the administration of his remaining associates, enjoys a very high standing in business circles and is the Agent in Brazil of the Munson Steamship Line, the American Express Company, the White Company, the Marine Office of America, and many other important American firms. In addition to representing American interests, the Company conducts on its own account

* Memoir prepared by A. Lund, Director, The Federal Express Co., Rio de Janeiro, Brazil.

a general express, transportation, storage, insurance, tourist, and banking business.

Mr. Stevenson was very active and had an extraordinary amount of executive ability which he utilized in his social as well as in his business relations. He was not only one of the founder members of the American Chamber of Commerce for Brazil, but was also its first Secretary. He served continuously and tirelessly in this capacity until after many years he was induced to accept the Presidency. He was also one of the founders of the Rio de Janeiro Country Club of which he served as Director and he was a member of the Jockey and Golf Clubs. His vision and initiative in establishing the Country Club made evident only one of the many ways in which he used his great ability for the pleasure as well as the profit of the entire community. He was deeply interested in furthering good relations between Brazilians and Americans, and he worked to this end with great success.

Mr. Stevenson died of heart disease, from which he had been suffering for a long time, at his home in Rio de Janeiro on November 8, 1928. His burial was attended by practically the entire American Colony and by the most prominent members of Brazilian society and of business firms of Rio de Janeiro. He is survived by his widow, Esther S. Stevenson, of Philadelphia, Pa.

His success in Brazil was due, to a great extent, to the fact that he made Rio de Janeiro his permanent home, took pains to study and understand the psychology of Brazilians, and to follow the Republic both in its material and intellectual aspect. He associated freely with Brazilians and, consequently, was able to promote the American business interests with which his Company was connected in a most efficient manner and without the friction which develops when there is lack of mutual understanding.

Mr. Stevenson was elected a Junior of the American Society of Civil Engineers on April 4, 1905, and an Associate Member on November 6, 1907.

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